The composition and uses of slurries in civil engineering practice.

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THE COMPOSITION AND USES OF SLURRIES IN CIVIL ENGINEERING PRACTICE

BY

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ABSTRACT

The physical properties of clay slurries are examined in relation to their use in oil well drilling and slurry trench work. These properties can be varied by chemical treating agents and the operation of these agents is discussed in terms of the chemistry of the various clay minerals.

The influence of a slurry on the permeability and consistency of a soil is investigated using a specially designed permeameter. Techniques for the measurement of permeability are also examined and the results from large scale crossflow and piezometer tests are compared with those from the permeameter tests.

The effect of a slurry in the voids of a gravel is then examined and it is found that the slurry can reduce the permeability of the gravel to about 10^{-6}cm/s. If a sufficiently high hydraulic gradient is applied to the slurry/gravel combination the slurry can be displaced and a theory is developed to relate this gradient to the particle size of the gravel.

The settlement of the solids from a slurry is investigated both in tubes and in the voids of a gravel and it is concluded that a slurry that can seal a gravel will not settle in the voids of that gravel.

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PHYSICAL PROPERTIES OF DRILLING MUDS AND CLAY SLURRIES

The fundamental properties required of drilling muds and slurries are:-

1. Viscosity, gel strength and thixotropy
2. Density
3. Filter cake formation.

1.1 Viscosity, gel strength and thixotropy

a Definitions

For flow parallel to a plane wall the fluid in contact with the wall is at rest and at other sections the fluid velocity is a function of the distance from the wall. Figure 1 shows an element of fluid in such a flow.

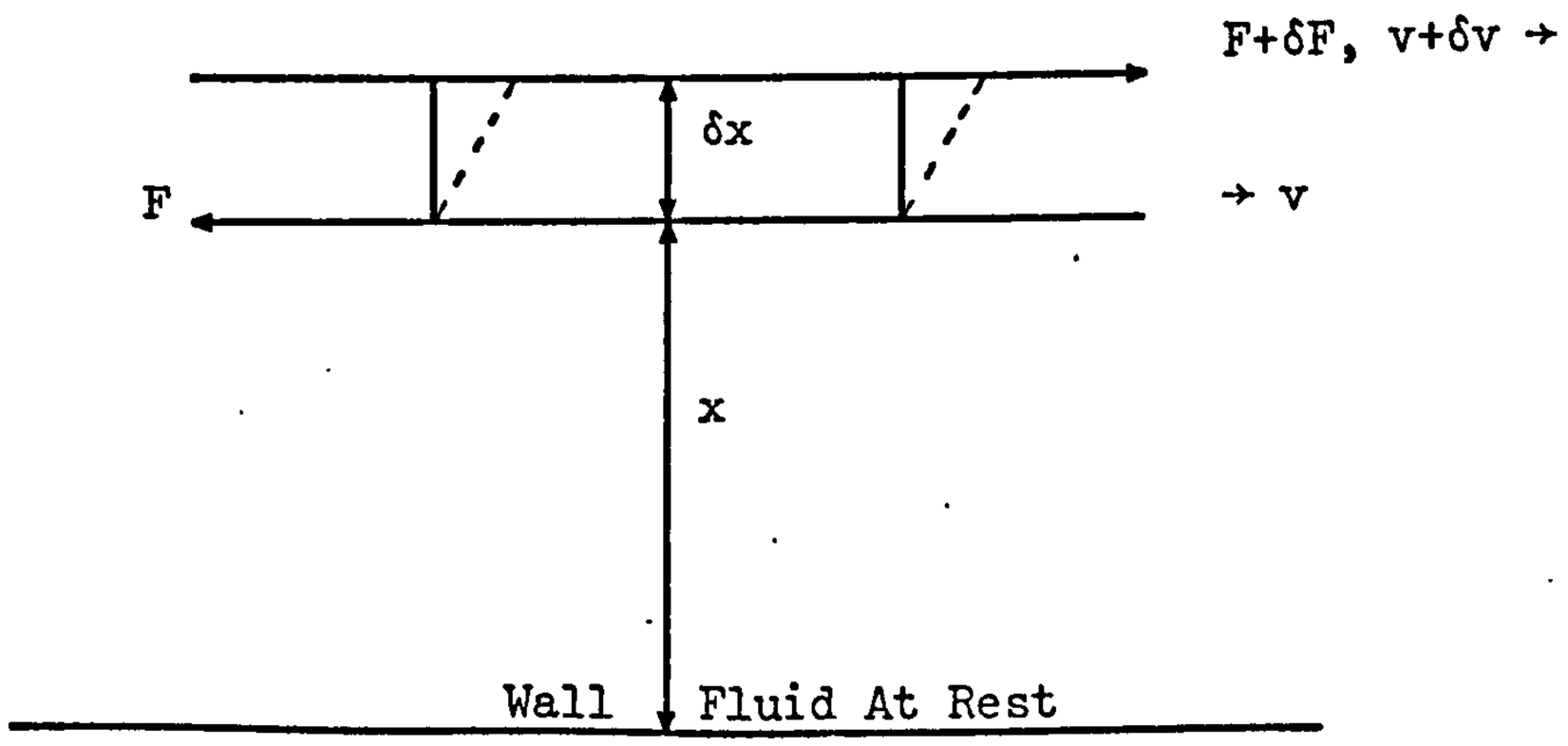


Figure 1

The velocity difference across the element will cause it to be sheared from a rectangle at one instant to a parallelogram (as shown by the dotted lines) a short interval of time later. If

the element is sheared through an angle θ in time t the rate of shear is θ/t

but $$\theta = \frac{dv}{dx}.t$$

$$\therefore \quad \frac{\theta}{t} = \frac{dv}{dx}$$

where v is the velocity of an element at a distance x from the wall.

Now if the force F applied to the element acts over an area A parallel to the wall the applied shear stress is F/A.

For many fluids the ratio of shear stress to rate of shear is found to be constant and is called the viscosity (μ)

Hence $$\mu = \frac{F/A}{\frac{dv}{dx}} \qquad (1.1)$$

Fluids which obey the equation are said to be ideal or Newtonian fluids.

Clay slurries in general exhibit more complex flow properties and Fig.2 shows a typical plot of Rate of shear against Shear stress for a slurry which has been vigorously stirred.

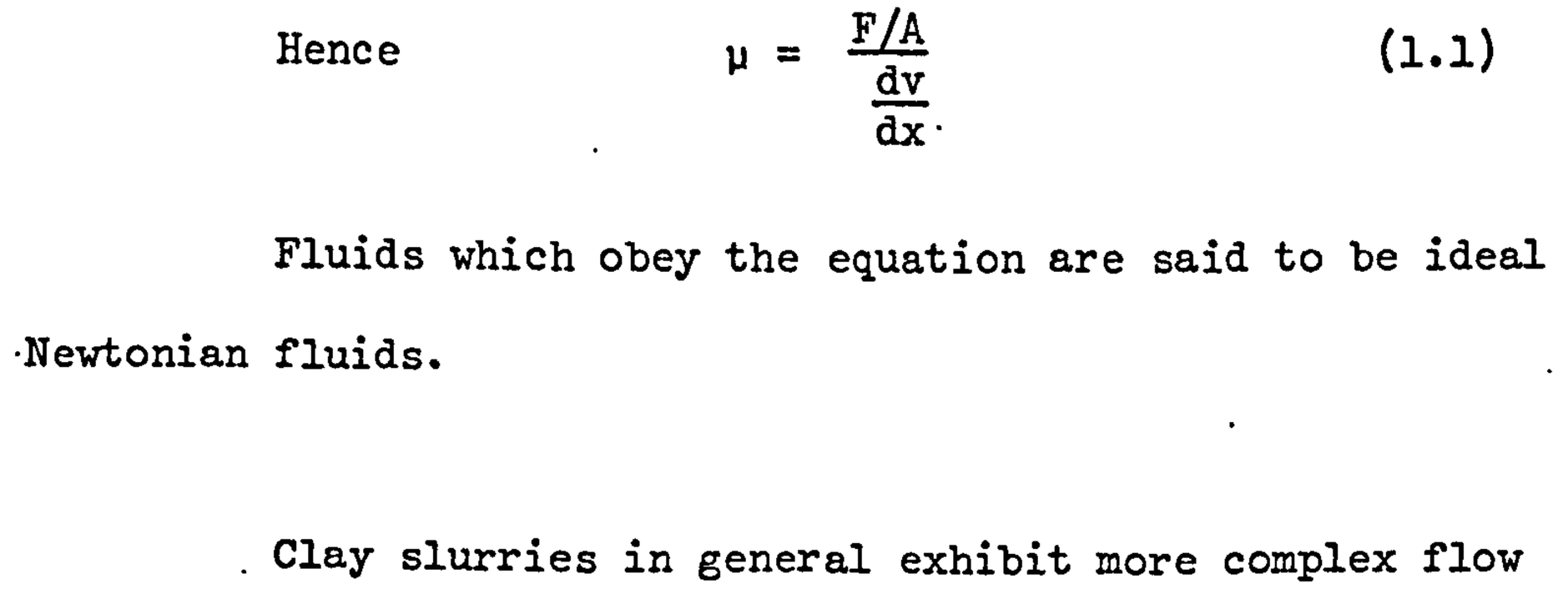


Figure 2

τ_s is the gel strength. It represents the minimum shear stress required to produce flow.

τ_o is the Bingham yield, the yield stress or yield point. It is the projection of the straight line portion of the curve on to the shear stress axis.

If τ_s and τ_o are coincident the slurry is said to be an ideal Bingham fluid or an ideal plastic fluid.

μ_a the apparent viscosity is the viscosity obtained by assuming the fluid to be Newtonian that is:-

$$\mu_a = \frac{OY}{OX}$$

μ_p the plastic viscosity or differential viscosity is defined as $d\tau/ds$ hence for the straight line section of the curve:-

$$\mu_p = \frac{BY}{AX}$$

The gel strength observed in clay suspensions is caused by linkage between the clay particles to form a structure, the nature of this linkage will be discussed in Chapter 2. When a shear stress is applied to such a suspension no flow occurs until the gel strength is reached, at this point the structure breaks but is not completely

destroyed. The degree of breakdown depends on the rate of shear and the time for which it is applied. If after the application of a shear stress the suspension is allowed to stand quiescent the gel structure will re-form, this reversible gelation is referred to as thixotropy. Fig.3 shows typical plots of Rate of shear against Shear stress for a thixotropic suspension.

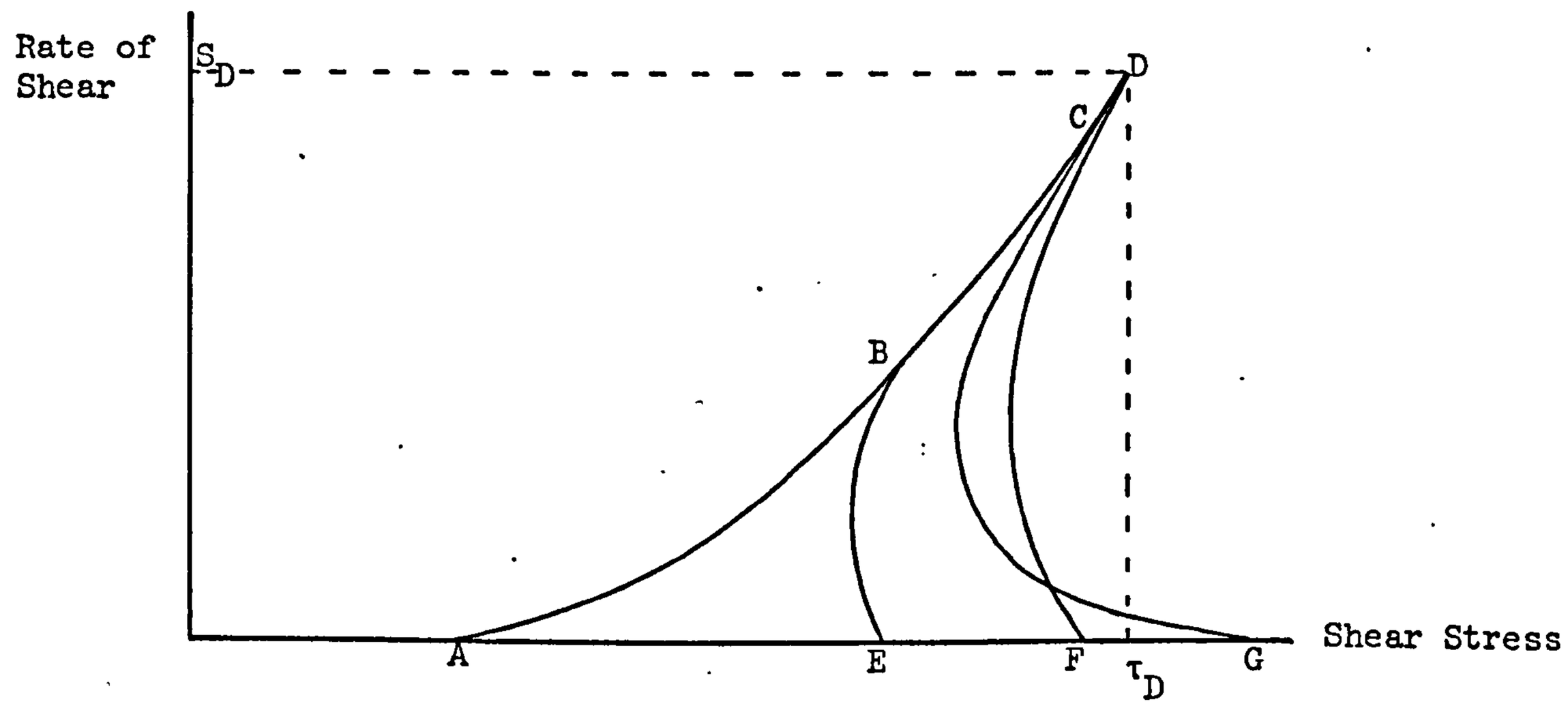


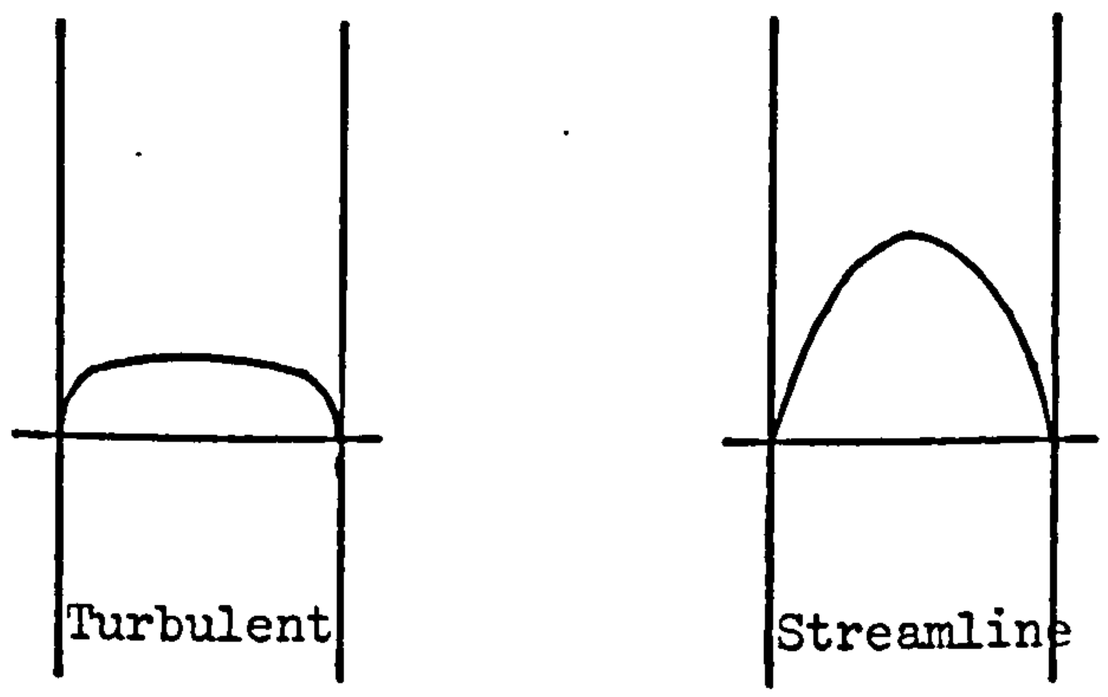
Figure 3

If a constant rate of shear S_D is applied to the suspension the shear stress will decrease until the equilibrium value τ_D is reached at which the rate of breakdown of the gel structure is equal to the rate of formation. The curve DCBA can then be obtained by rapidly reducing the rate of shear so that the gel structure does not have time to re-form. If when the point A has been reached the suspension is allowed to stand quiescent for some rest time t and the shear rate then rapidly increased, so that the structure does not have time to breakdown, the curve FCD will be obtained, if some shorter rest time is allowed the curve EBCD will result. The cuves DCBA, FCD and EBCD can

only be obtained with a recording viscometer or by laborious point by point determination with the shear rate S_D applied between each reading and an appropriate rest time allowed. As a result, in practice normally only the equilibrium curve GCD, obtained by allowing sufficient time for equilibration at each shear rate, and the points A and F (with a rest time of ten minutes) are measured. The shear stress at A is referred to as the initial gel strength and at F as the ten minute gel strength. The difference between F and A gives some indication of the thixotropy though no quantitative definitions are used.

b <u>Specifications for Drilling</u>

Fig.4 shows typical design curves for drilling muds as recommended by Milchem (1966)*. In drilling, low viscosity and gel strength are of importance in connection with removal of drill cuttings from the hole. For optimum removal rate the flow should be turbulent (Fig.5a). If streamline flow (Fig.5b) occurs the low velocity at



<u>Figure 5</u> Velocity profiles for pipe flow

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* References are listed alphabetically in the bibliography.

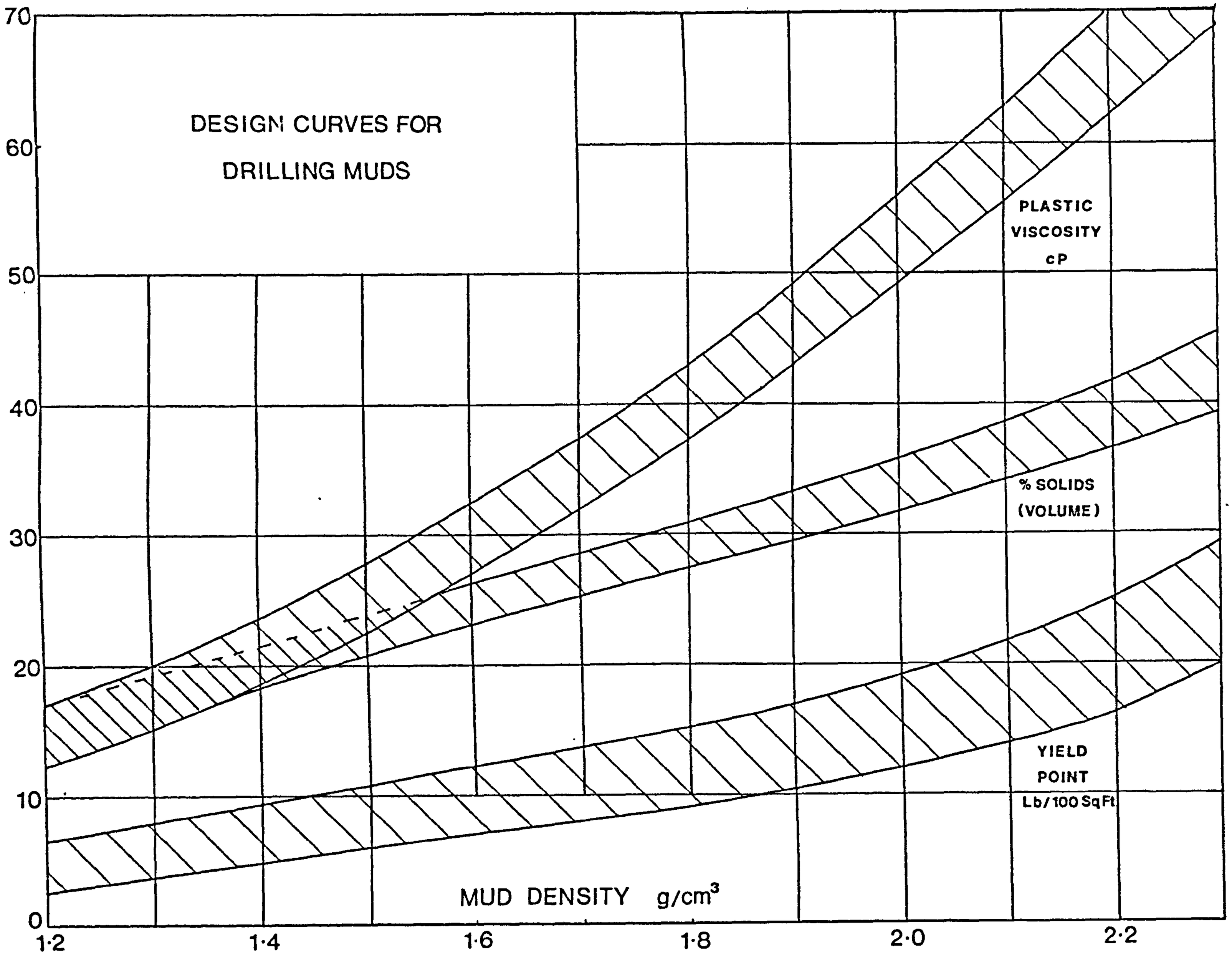
DESIGN CURVES FOR
DRILLING MUDS
PLASTIC
VISCOSITY
cP
% SOLIDS
(VOLUME)
YIELD
POINT
Lb/100 SqFt
MUD DENSITY g/cm³
70
60
50
40
30
20
10
0
1·2
1·4
1·6
1·8
2·0
2·2

Fig. 4

the wall may cause cuttings to stick. The flow rate at which transition from streamline to turbulent flow occurs decreases as the viscosity decreases. Thus viscosities are kept as low as is possible consistent with providing sufficient drag force to remove cuttings from the hole.

Initial gel strengths are also kept as low as possible (0-2† lb /100sq.ft) as they serve no useful purpose and waste pumping power. Some thixotropic gel strength (10 minute gel strengths in the range 5-25 lb /100sq.ft) is required to prevent settlement of cuttings and weighting material if the circulation is stopped. Though the gel must not be so strong that it cannot be broken by the pressure available from the circulation pumps.

c Specifications for slurry trenches

Very little information is available about the design criteria for slurries in trenching work, though the following points must be considered:-

(i) The viscosity and initial gel strength should be low so that spoil dropped from the grab during digging is not held in suspension: otherwise the slurry will rapidly become very thick.

(ii) To prevent wastage the slurry must drain freely from the grab as it is raised from the trench.

(iii) If a high slurry density is required the initial gel strength may be increased so that a limited amount of spoil retention occurs. This may require close control of the slurry if it is not to become

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† As recorded on the Fann V-G meter this corresponds to 0.96 N/m^2.

too thick, and if very high densities are required it is probably best to use a weighting agent.

(iv) In porous formations the slurry must have sufficient gel strength to seal the pores. Some thixotropic increase in gel strength may also be an advantage as it will help to bind loose sand or gravel which are often associated with such formations.

In general some compromise between these requirements will be necessary and specifications are often worded to allow considerable variation in the slurry properties. Typical specifications would be those required for the bentonite slurries at the Duncan Dam (Duguid, Forbes, Gordon and Simmons, 1971) and the Wanapum Development (La Russo, 1963). At the Duncan Dam a minimum apparent viscosity of 15cP was required. At Wanapum a viscosity of 15cP was required at the start of digging and 30cP at the start of backfilling. For bentonite an apparent viscosity of 15cP probably corresponds to a yield point of 15-20 lb /100sq.ft though no requirements as to gel strengths were made in either specification.

Bentonite is normally used for slurry trenches and concentrations range from 3-10% (Braun, 1969) but even this gives no indication of the initial or thixotropic gel strengths used as the properties of such slurries can show wide variations. For example the data given in Table I have been obtained for bentonites at a concentration of 6%*. The figures for London tap water are given for comparison.

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* all slurry concentrations are given in g /100g of slurry.

TABLE I

Source of clay	Plastic viscosity	Initial gel strength	10 minute gel strength
	cP	lb /100sq.ft	lb /100sq.ft.
Wyoming	16	0	0.5
England	12	32	87
England	6	37	73
Wyoming (polymer treated)	23	113	>200
Cyprus	3	1	1
London tap water	1	0	0

d Measurement of flow properties

Van Wazer, Lyons, Kim and Colwell (1963) give a full review of methods available for the measurement of flow properties. Only two methods will be considered here in detail.

(i) Measurement of flow rates and pressure drops in pipes.

Consider an ideal plastic fluid which has an equation of flow:-

$$\tau - \tau_o = \mu_p S \qquad (1.2)$$

where

τ is the shear stress

τ_o is the yield stress

S is the rate of shear

μ_p is the plastic viscosity.

If this fluid is flowing in a pipe of radius R and length L for an annular streamline of radius r and velocity u

$$S = \frac{du}{dr} \qquad (1.3)$$

and

$$\tau = \frac{r\Delta P}{2L} \qquad (1.4)$$

where ΔP is the pressure drop in length L

hence equation (1.2) becomes

$$\frac{r\Delta P}{2L} - \tau_o = \mu_\rho \frac{du}{dr} \qquad (1.5)$$

It can be seen from equation (1.4) that τ increases from zero at the centre to a maximum at the wall, hence there will be a plug of radius r_o at the centre of the pipe which will behave as a solid since $\tau < \tau_o$

where

$$r_o = \frac{2\tau_o L}{\Delta P}$$

Fig.6 shows the velocity profile for such a flow. As ΔP increases r_o will decrease but will not vanish.

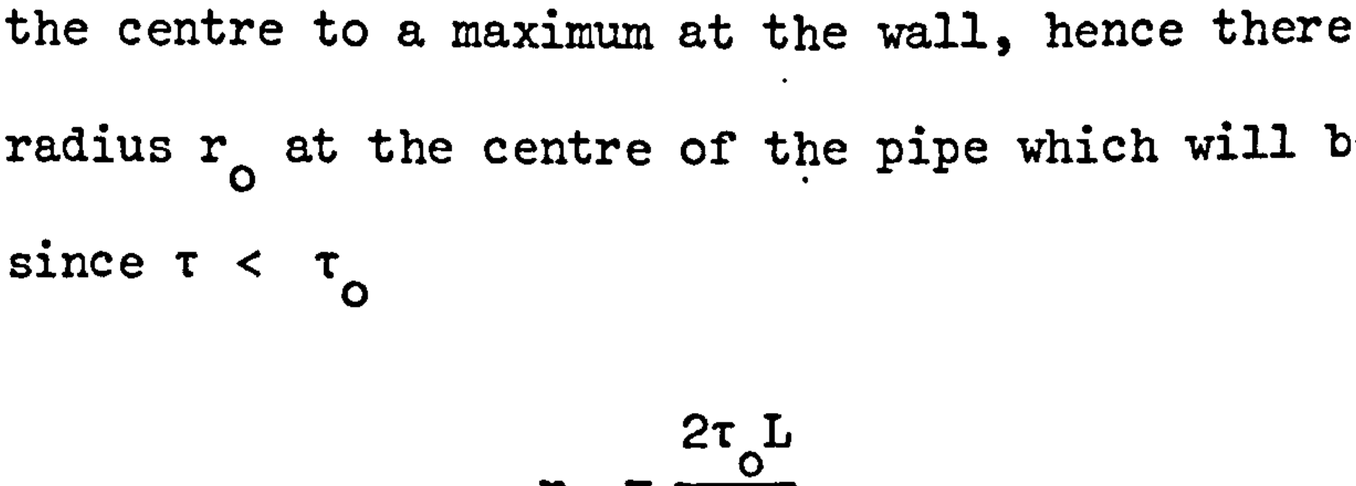


Figure 6

The total flow, Q through the pipe can be found by dividing the flow profile by planes of thickness du perpendicular to the flow axis.

Hence

$$Q = \int_{r_o}^{R} \pi r^2 du$$

Substituting from equation (1.5)

$$Q = \frac{\pi}{\mu_p} \int_{r_o}^{R} \left(\frac{r^3 \Delta P}{2L} - r^2 \tau_o \right) dr$$

whence

$$Q = \frac{\pi R^4 \Delta P}{8 L \mu_p} \left| 1 - \frac{4}{3}\left(\frac{2\tau_o L}{\Delta P R}\right) + \frac{1}{3}\left(\frac{2\tau_o L}{\Delta P R}\right)^4 \right| \qquad (1.6)$$

This equation is known as the Buckingham-Reiner equation (Reiner, 1949) and can be transformed in terms of the variables

$$S = \frac{4Q}{\pi R^3}$$

which has the dimensions of Time^{-1} and hence can be considered as a rate of shearing

and

$$\tau_R = \frac{R \Delta P}{2L}$$

The shear stress at the wall

hence

$$S = \frac{1}{\mu_p}\left|\tau_R - \frac{4}{3}\tau_o + \frac{1}{3}\frac{\tau_o^4}{\tau_R^3}\right| \qquad (1.7)$$

Thus a plot of S against τ_R will have the form shown in Fig.7.

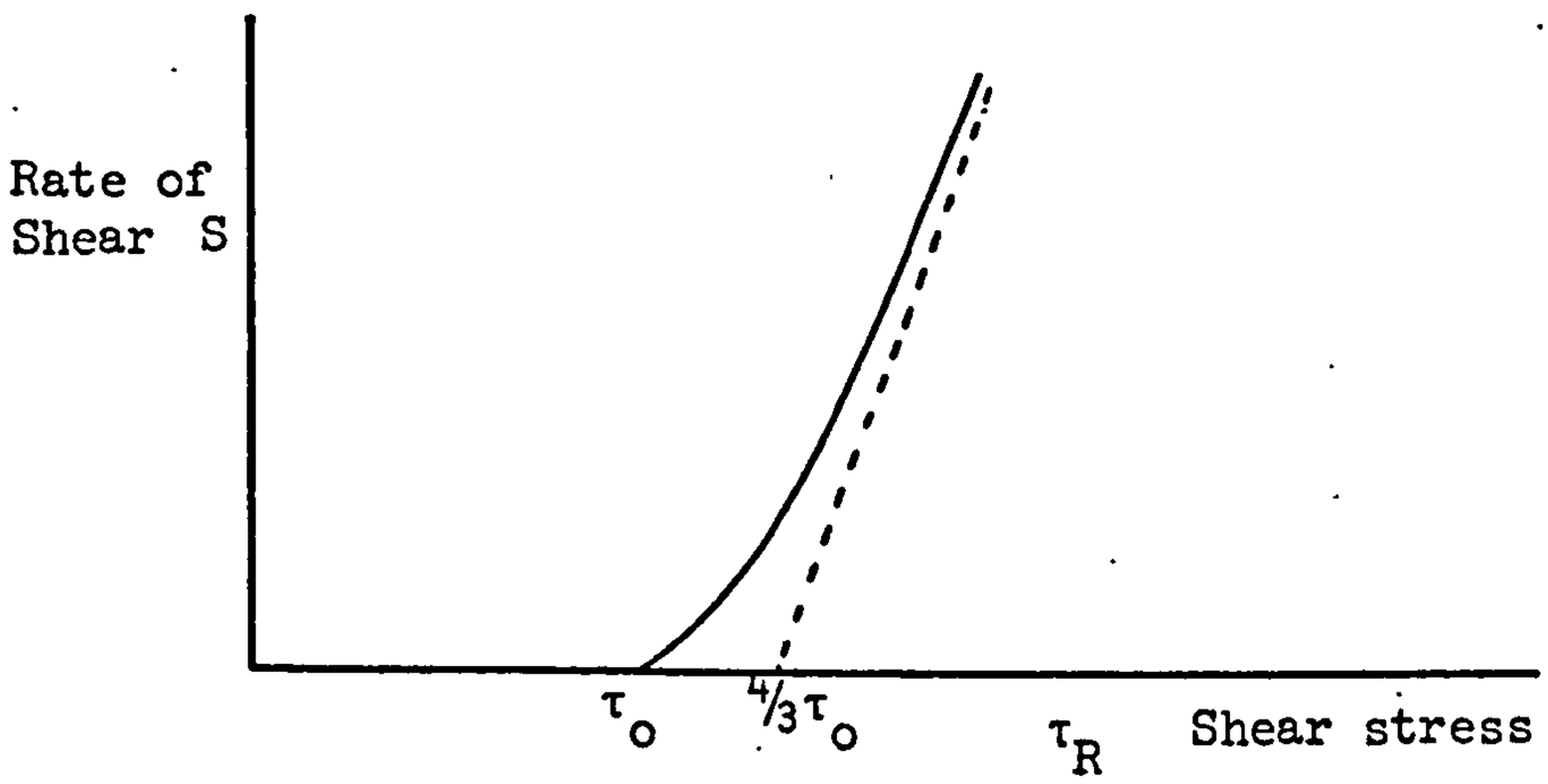


Figure 7

at S = 0, $\tau_R = \tau_o$ hence the intercept of the curve is equal to the yield stress of the fluid. For large τ_R, τ_o^4/τ_R^3 tends to zero and the curve approximates to a straight line. The intercept produced by extrapolating this "straight line" to the shear stress axis is $^4/_3\ \tau_o$. The curvature of the S against τ_R plot is due to the gradual transition from plug flow to streamline flow and does not imply that the plastic fluid is non ideal. For clay slurries which behave as non ideal plastic fluids the curvature is more pronounced and the intercept of the curve is called the gel strength, τ_S(Fig.2) and there is no simple relationship between this and the yield stress τ_o, though the intercept of the "straight line" can be taken as $^4/_3\ \tau_o$.

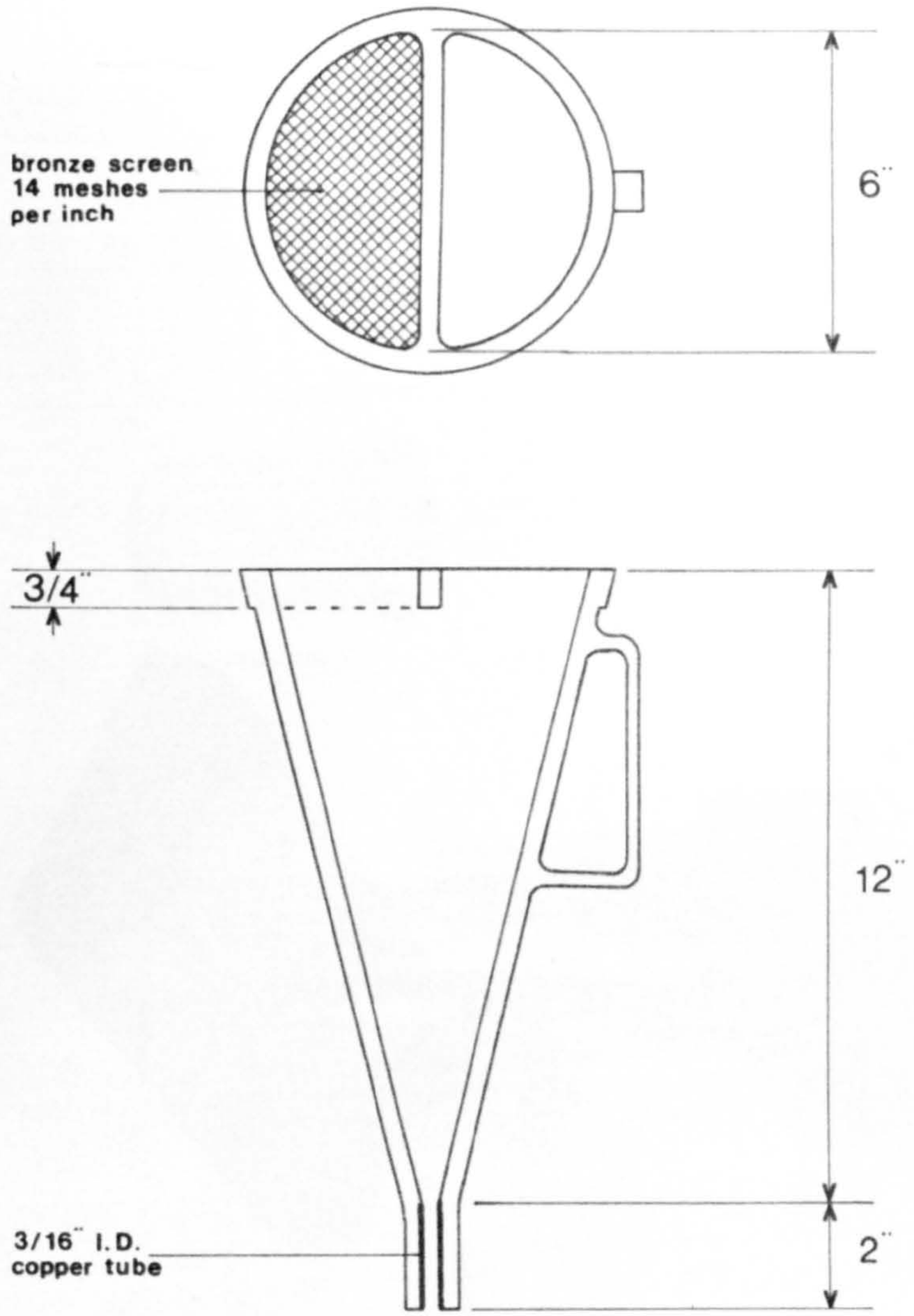
Thus for slurry in a pipe τ_S can be found by measuring the minimum pressure required to produce flow, and μ_p and τ_o can be found by measuring the flow rate at various pressure gradients, though this is considerably more laborious and the calculations are more often used in reverse to calculate the flow rate knowing μ_p and τ_o.

The Marsh Funnel (Fig.8) is a very simple viscometer based on the measurement of flow rates. The instrument consists of a funnel of standard dimensions which is filled with 1500 cm^3 of slurry through a wire screen to remove coarse material. The time for 946 cm^3 (1 US quart) or sometimes 1000 cm^3 to flow from it is then measured and is quoted as a viscosity in seconds for the flow quantity. Thus water at 21°C has a viscosity of 26 seconds for 946 cm^3. The measured viscosity for a slurry is considerably influenced by its rate of gelation, and also by density which affects the hydrostatic head in the funnel. Because of these factors the viscosities obtained with the Funnel cannot be directly correlated with those obtained from other instruments, but it does provide a useful routine comparative test on site.

(ii) <u>Rotational Viscometers</u>

There are numerous types of rotational viscometer but the Fann V-G meter (Fig.9) has been accepted as the standard instrument in drilling and is also generally accepted in slurry trench work.

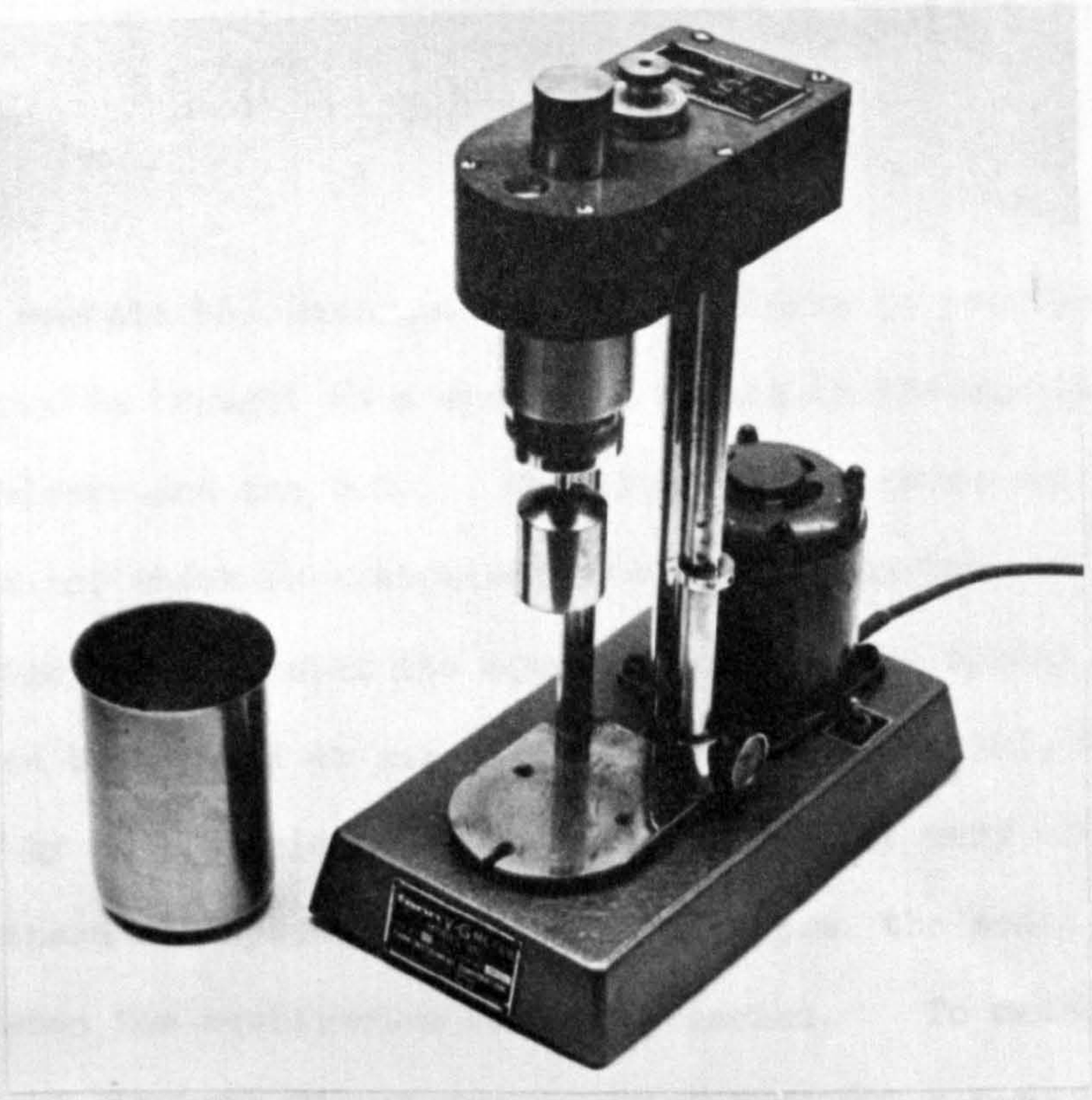
Funnel and graduated receiver



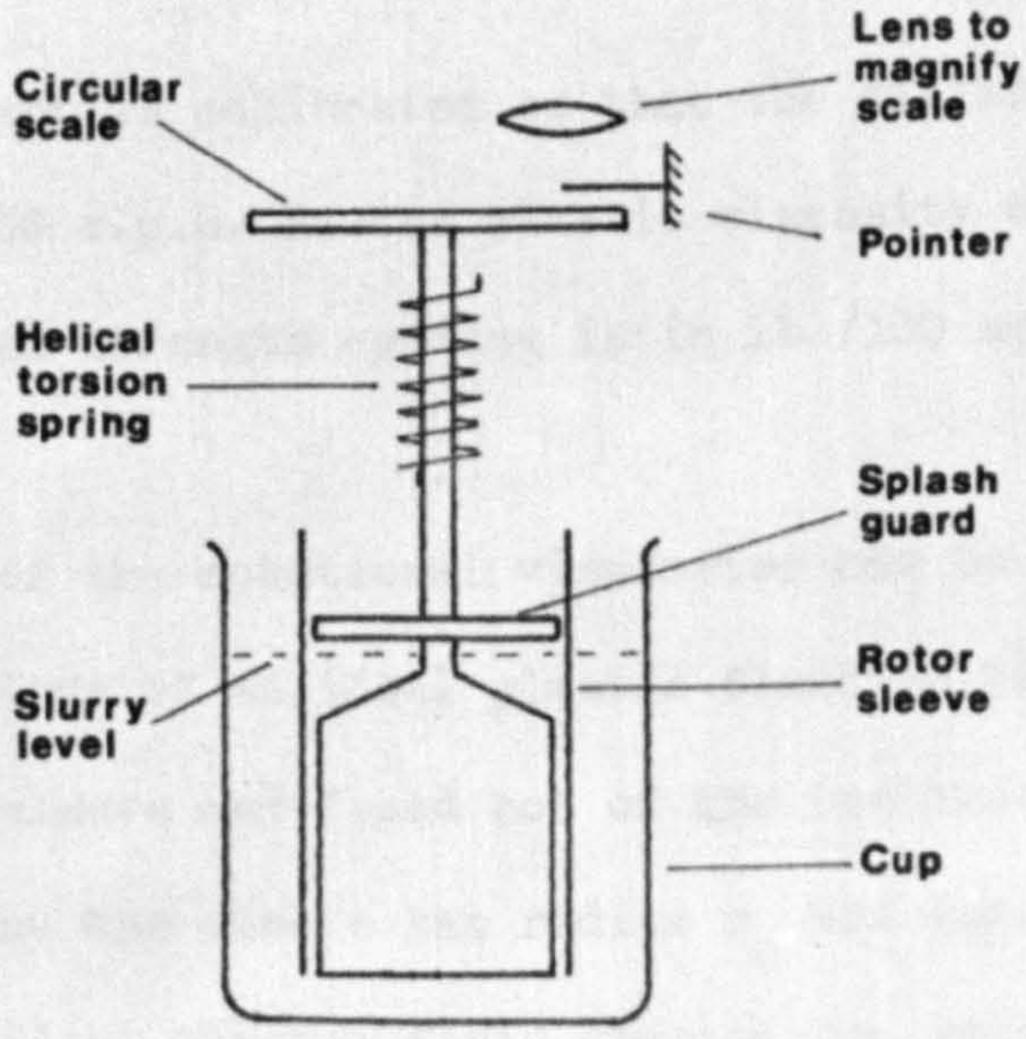
Funnel dimensions

Fig. 8

THE MARSH FUNNEL



Meter with sleeve removed



Schematic diagram

Fig. 9 THE FANN V-G METER

To operate the instrument a cup of slurry is positioned so that the slurry is brought to a specified height in the annular space between the sleeve and the bob. Rotation of the sleeve exerts a torque on the bob which is restrained by a torsion spring and a scale attached to the bob indicates its movement against the spring restraint. The sleeve can be rotated at six different speeds (600, 300, 200, 100, 6, 3 r.p.m.) by an electric motor and also by hand at very slow speeds. For each of these six speeds starting at 600 r.p.m. the scale reading is recorded when the equilibrium value is reached. To measure the initial gel strength the sleeve is rotated at 600 r.p.m. until equilibrium is reached, then stopped and rotated very slowly by hand, the maximum reading recorded before the gel breaks is the initial gel strength. The 10 minute gel strength is obtained similarly but 10 minutes is allowed to elapse between stirring and reading.

The instrument is calibrated so that the difference in the torque readings at 600 and 300 r.p.m. is the plastic viscosity directly in centipoise, and the gel strength reading is in lb /100 sq.ft.

The theory of the rotational viscometer can be developed by considering the behaviour of an ideal plastic fluid in the annular space between the rotating sleeve and fixed bob of the instrument. If the bob has a radius r_b and the sleeve has radius r_s and angular velocity w and the slurry is an ideal plastic fluid obeying the equation (1.2). For small values of w the fluid in contact with the bob will be liquid

but $\tau < \tau_o$ for $r > r_o$ if r_o is defined by:-

$$r_o^2 = \frac{M}{\tau_o 2\pi h} \qquad (1.8)$$

where

M is the torque on the bob

h is the height of the bob.

Hence the fluid in the annulus $r_o - r_s$ will behave as a solid as shown in Fig.10

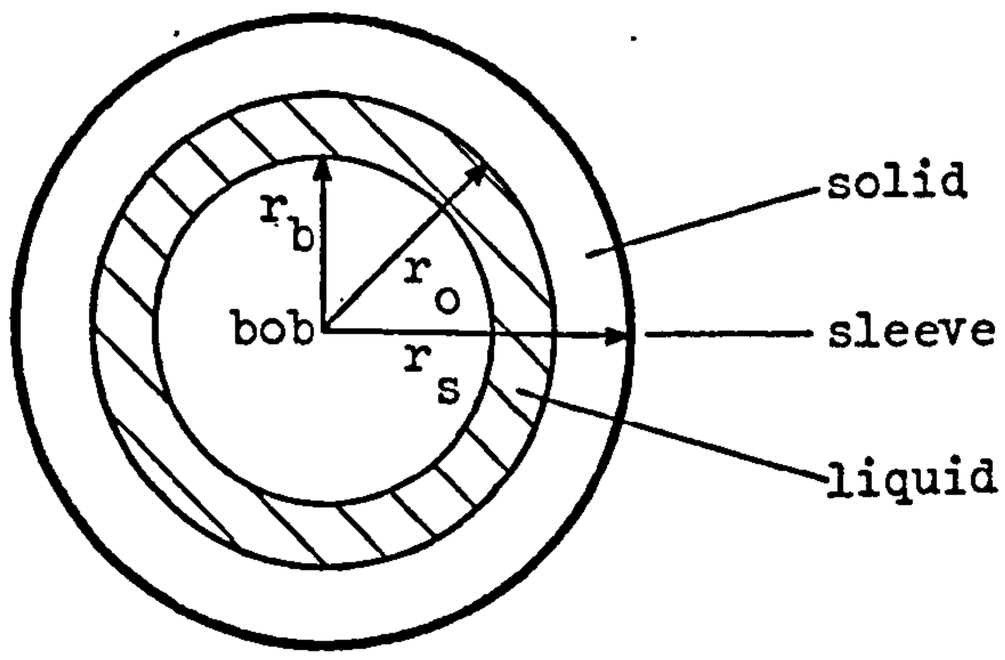


Figure 10

The rate of shear, S, is given by:

$$S = r \frac{d\theta}{dr}$$

where

$\dot{\theta}$ is angular velocity of the fluid at radius r

and the shear stress at radius r is:-

$$\tau = \frac{M}{2\pi h r^2}$$

substituting for S and τ equation (1.2) yields:-

$$\mu_p r \frac{d\dot{\theta}}{dr} = \frac{M}{2\pi h r^2} - \tau_o \qquad (1.9)$$

integrating and fitting the boundary condition $\dot{\theta} = 0$ at $r = r_b$

$$\mu_p \theta = \frac{M}{4\pi h}\left(\frac{1}{r_b^2} - \frac{1}{r^2}\right) - \tau_o \ell n \frac{r}{r_b} \qquad (1.10)$$

Now from equation (1.8) if:-

$$2\tau_o \pi h r_b^2 < M < 2\tau_o \pi h r_s^2$$

there will be a ring of solid as shown in fig.10 and all the material between r_s and r_o will rotate with angular velocity w thus from equation (1.10):

$$\mu_p w = \frac{M}{4\pi h}\left(\frac{1}{r_b^2} - \frac{1}{r_o^2}\right) - \tau_o \ell n \frac{r_o}{r_b} \qquad (1.11)$$

However if:

$$M > 2\tau_o \pi h r_s^2$$

all the fluid will behave as liquid and:

$$\mu_p w = \frac{M}{4\pi h}\left(\frac{1}{r_b^2} - \frac{1}{r_s^2}\right) - \tau_o \ell n \frac{r_s}{r_b} \qquad (1.12)$$

Thus equations (1.11) and (1.12) represent the flow curves for an ideal plastic fluid in a rotational viscometer. Fig.11 shows a plot of w against M for the Fann V-G meter.

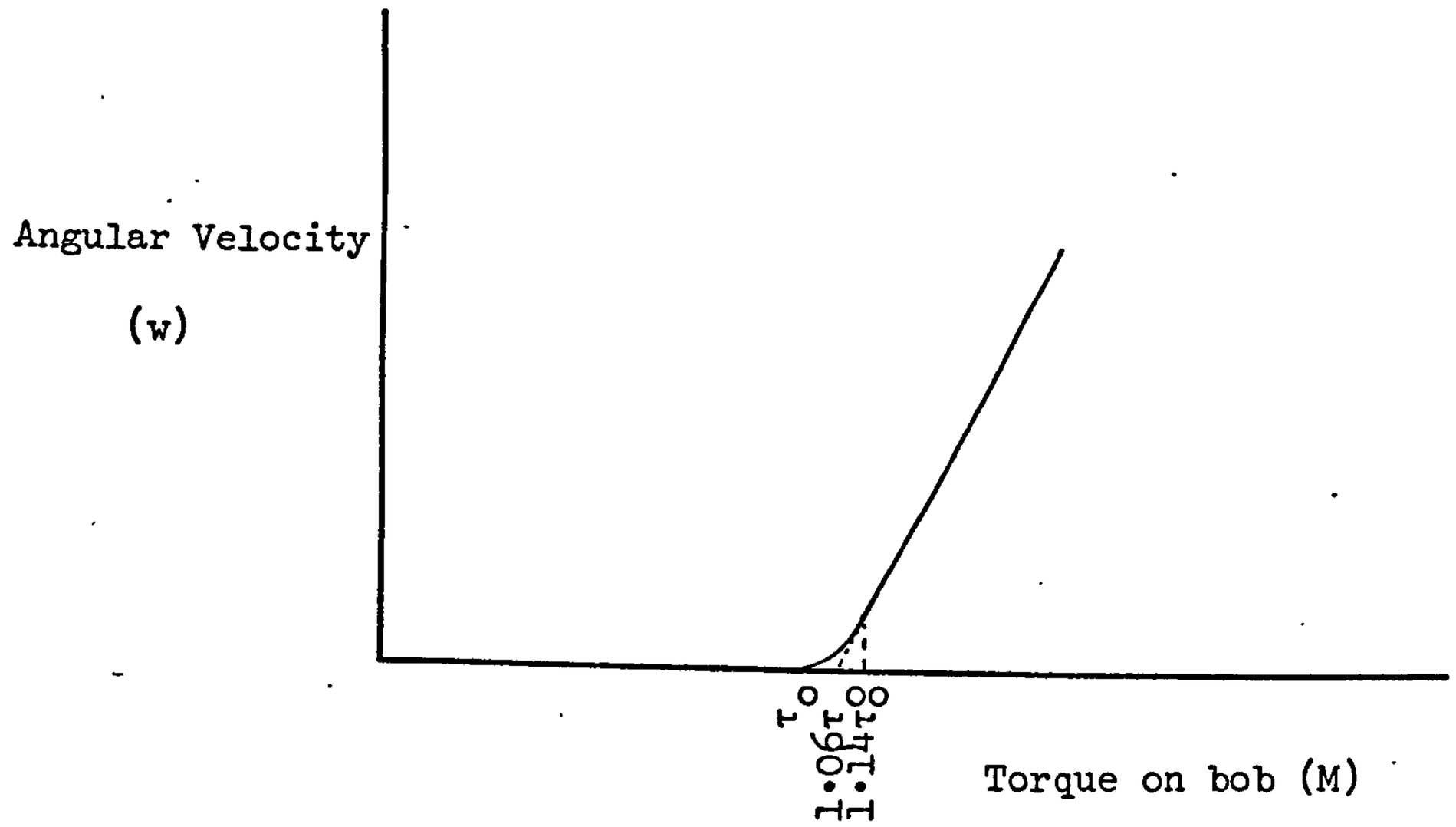


Figure 11

The curved section corresponds to equation (1.11) and the straight line to equation (1.12). The intercept for straight line is given by:

$$M \doteq 4\pi h\tau_o\left(\frac{1}{r_b^2} - \frac{1}{r_s^2}\right)^{-1} \ell n \frac{r_s}{r_b}$$

which if the values of r_s and r_b for the Fann V-G meter are substituted gives:

$$M = 1{\cdot}06\tau_o .$$

Thus for all practical purposes the curved section can be ignored for the Fann V-G meter and the intercept can be taken to be the yield stress.

As for pipe flow the curvature is more pronounced for practical slurries (Fig.12),

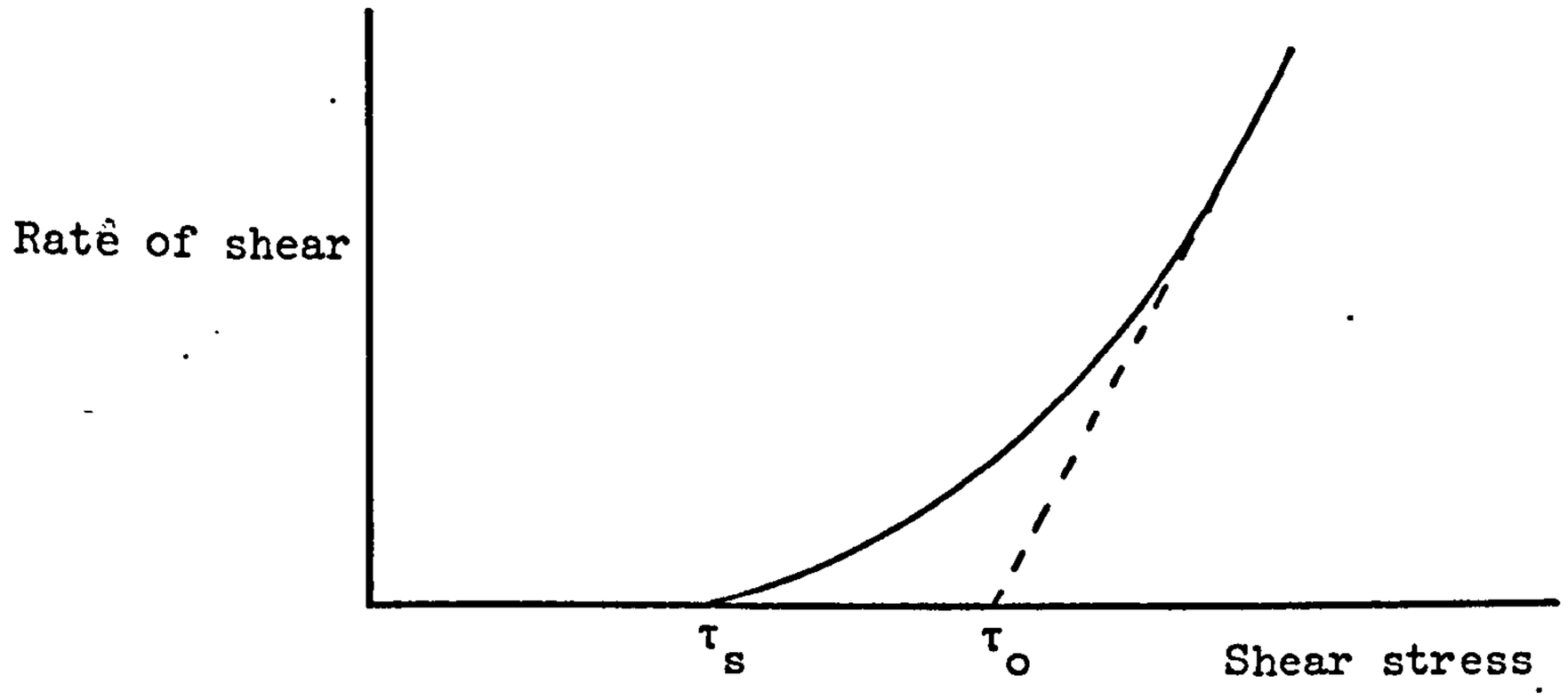


Figure 12

and there is no simple relationship between τ_s and τ_o.

1.2 Density

a Specifications

In drilling as in slurry trench work the hydrostatic pressure of the slurry acting against the walls of the hole must be sufficient to prevent collapse. Additionally in drilling the pressure must be high enough to prevent blow-outs if abnormal gas pressures are encountered.

In drilling where very high densities are sometimes required, barytes (Barium sulphate) with a density of 4.2 g /cm^3 is used as a weighting agent. In slurry trenches weighting agents are not often used as the densities required seldom exceed those achieved during digging

as the slurry picks up material from the excavation. Typical initial densities would be in the range 1.01-1.09g /cm^3 (63-68 lb/ft^3) corresponding to 3-10% bentonite suspensions, and the final density could be as high as 1.52 g /cm^3 (95 lb /ft^3).

To predict the densities required in slurry trenches Nash and Jones (1963) developed formulae for the factors of safety of slurry filled trenches in clays and sands, and these give reasonable values if the predicted densities are taken as those after digging rather than the initial densities. The formulae neglect arching which may in practice provide an additional factor of safety as slurry trenches are often excavated in short panels.

b Measurement

In the laboratory densities are measured by weighing a known volume of slurry. On site a specially designed beam balance of the type shown in Fig.13 is often used.

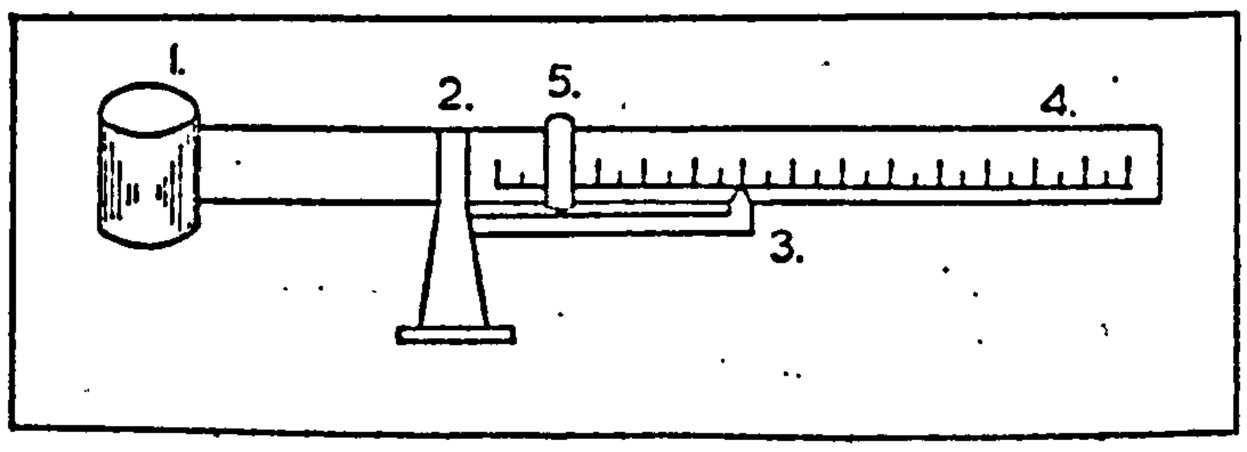


Figure 13 Mud balance.
1. Constant volume cup for slurry
2. Fulcrum for beam balance
3. Density indicator
4. Beam balance
5. Sliding weight

1.3 Filter Cake Formation

The slurry must be capable of forming a thin filter cake of low permeability over the walls of the hole. The cake must be thin so that it is not easily broken by tools in the hole. Low permeability is required to ensure that the full hydrostatic head of the slurry acts against the walls. Any fluid lost will not only deplete the slurry but may also cause softening of clays or swelling of shales in the formation with consequent reduction in wall stability. When digging through porous formations the slurry must gel in the void spaces and have sufficient gel strength to withstand the hydrostatic pressure in the trench. In very porous formations fibrous material may be added to assist the sealing.

a Testing and Specification

The standard filter press was developed for the drilling industry and the results are difficult to interpret in terms of permeabilities.

Fig.14 shows an exploded view of the American Petroleum Institute standard cell.

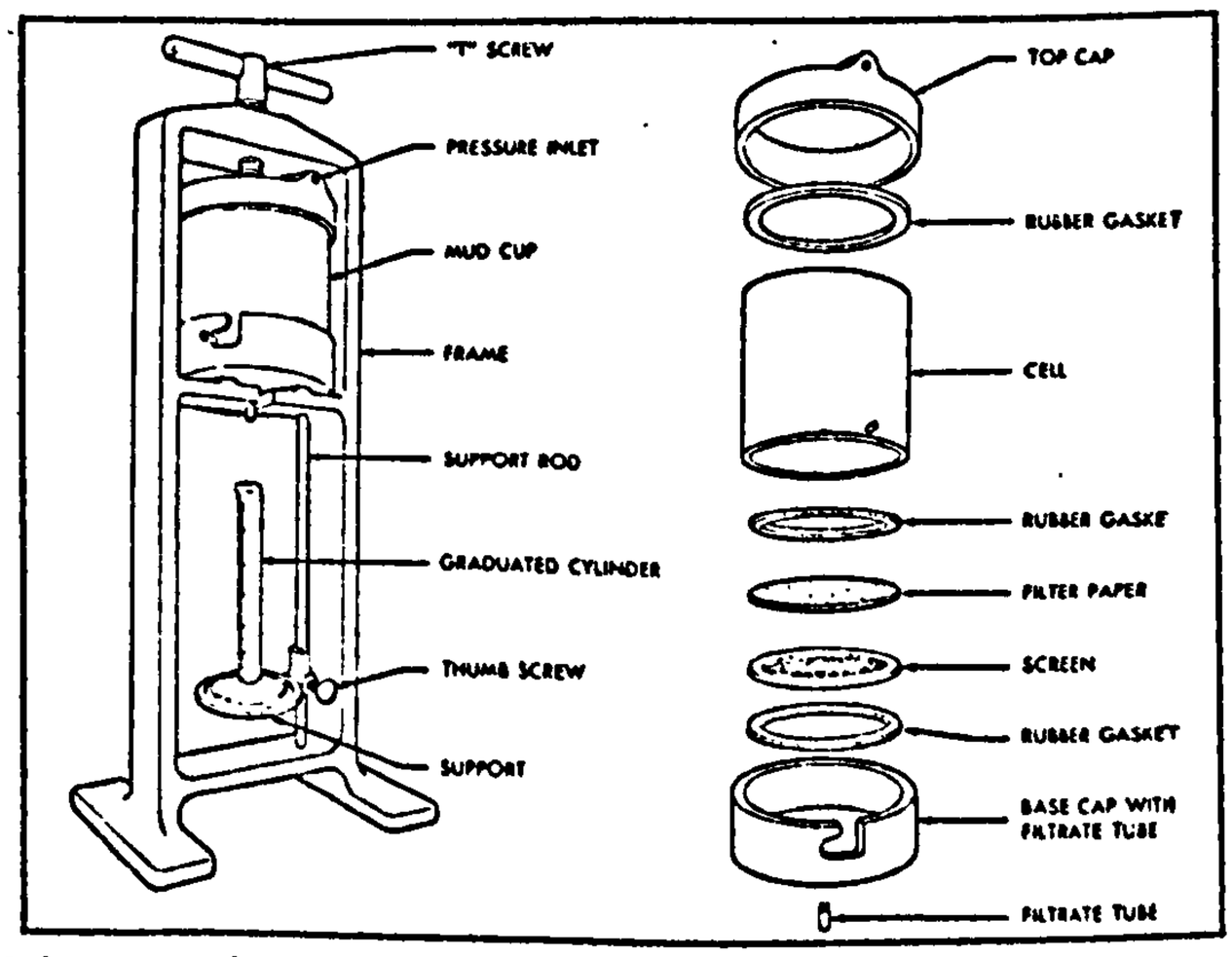


Figure 14 Standard Filter Press and Mud Cell Assembly

To perform a test the base cap is fitted to the cell with a Whatman No.50 filter paper covering the screen and the unit is filled with slurry to within a $\frac{1}{4}$" of the top. The top cap is then fitted and the whole unit placed in the frame and the T screw tightened. A 100 psi pressure is applied through the pressure inlet and the quantity of filtrate collected in 30 minutes is recorded. The filter cake thickness is measured after the excess slurry has been poured off and the cake gently washed.

The recommended filter loss for drilling fluids is less than $5cm^3$ in 30 minutes. This probably corresponds to a permeability of about 10^{-7} cm/s, though the test method prevents exact calculation. For slurries filter losses in the range of 20-$30cm^3$ are often specified.

CHAPTER TWO

THE STRUCTURE AND CHEMISTRY OF CLAYS

2.1 The Composition of Clays

2.2 The Clay Minerals

2.3 The Origin of Clays

2.4 The balance of attractive and repulsive forces on clay particles

(a) Face charges

(b) Edge charges

2.5 Particle Association

(a) Terminology

(b) The effect on flow properties

2.6 Chemical Treating Agents

2.7 Clays used in drilling and slurry trenches

2.8 Analysis of attapulgite and bentonite

THE STRUCTURE AND CHEMISTRY OF CLAYS

2.1 Composition of Clays

Clays consist of minute particles, typically less than a micron (10^{-4}cm) across, produced from the weathering of rocks. Chemical reactions of these particles are generally simple exchange reactions. For example an atom close to the surface of the particle say a sodium atom, might be replaced by a calcium atom. These reactions though chemically very simple have profound effects on the physical properties of clays.

To understand the relationship between the physical and chemical properties of clays it is necessary to consider the chemical structures of the clay minerals.

There are two principal building elements for clays. The first element referred to as the octahedral layer (Figure 15) consists of two closely packed sheets of oxygen atoms with much smaller metal atoms,

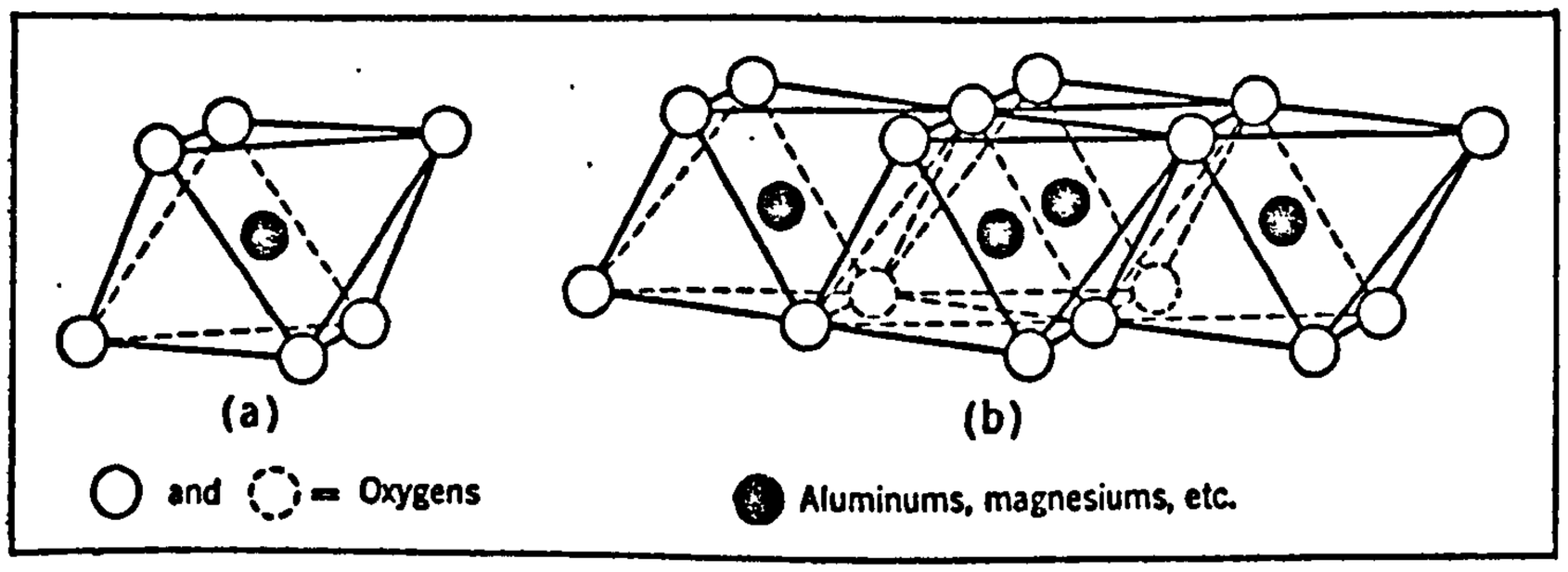


Figure 15 Diagrammatic sketch showing
(a) a single octahedraul unit and
(b) the sheet structure of the octahedral units.

normally magnesium or aluminium, embedded in the holes between the sheets. Each metal atom is arranged so that it is equidistant from six oxygen

atoms. It can be seen from Fig. 15a that the oxygen atoms are distributed round the metal atoms in the shape of an octahedron; the metal atom is said to be octahedrally co-ordinated by oxygen and it is this that gives the layer its name. When magnesium (Mg^{++}) is present all the holes between the layers are filled and the structure is said to be trioctahedral, but for Aluminium (Al^{+++}) only two thirds of the holes may be filled if electrical neutrality is to be preserved and the structure is said to be dioctahedral. The magnesium or aluminium can be replaced by iron (Fe^{++} or Fe^{+++}) and the oxygens (O^{--}) can be replaced by hydroxyls (OH^{-}) and the layer is not always electrically neutral.

The second building element shown in Fig.16 consists of silicon-oxygen tetrahedra and is referred to as the tetrahedral layer.

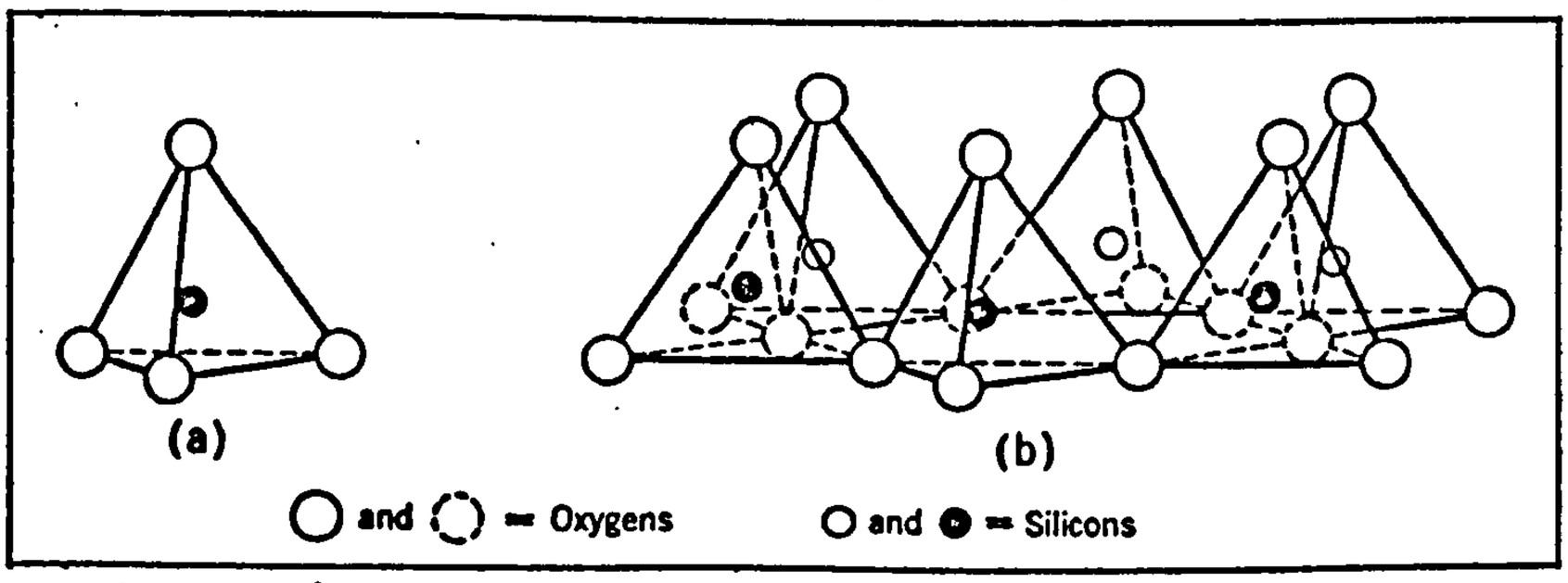


Figure 16 Diagrammatic sketch showing
(a) a single silica tetrahedron and
(b) the sheet structure of silica tetrahedrons arranged in a hexagonal network.

In each tetrahedron the silicon atom is equidistant from four oxygen atoms. The tetrahedra are arranged so that all the vertices point in the same direction and they are linked at the bases to form a layer. Once again atomic substitutions can occur, hydroxyl for oxygen and aluminium for silicon, and the layer may be electrically charged.

The spacing of the oxygen atoms in the octahedral and tetrahedral layers is very similar and this means that oxygen atoms can be shared between the layers without excessive distortion of the structure. The protruding oxygen atoms at the vertices of the silica tetrahedra are shared by the octahedral layer, if only one of the oxygen sheets in the octahedral layer is shared by a tetrahedral layer the minerals formed are referred to as two layer minerals. If both oxygen layers are shared three layer minerals are formed.

Fig. 17 shows the idealised structures of the two and three layer minerals. It can be seen from the charges associated with the atoms that some substitution of hydroxyl for oxygen must occur in the octahedral layer if the octahedral-tetrahedral units are to have no net charge though the layers themselves do carry a charge. In actual clay minerals the substitution is often more complex so that the units carry a net negative charge.

Clay particles are produced by stacking the two or three layer units one upon another. The bonding between the layers is by strong chemical bonding but between units many, generally weaker, types of bonding can occur. The nature of these bonds varies with the different clay minerals but a gravitational type bond known as Van der Waal's bonding is common to all minerals. The Van der Waal bond always exists between atoms but is exceedingly small and decays very rapidly with separation of the atoms, but for groups of atoms the forces are additive and decay less rapidly and thus may produce a weak but significant bonding between clay units. The thickness of clay particles

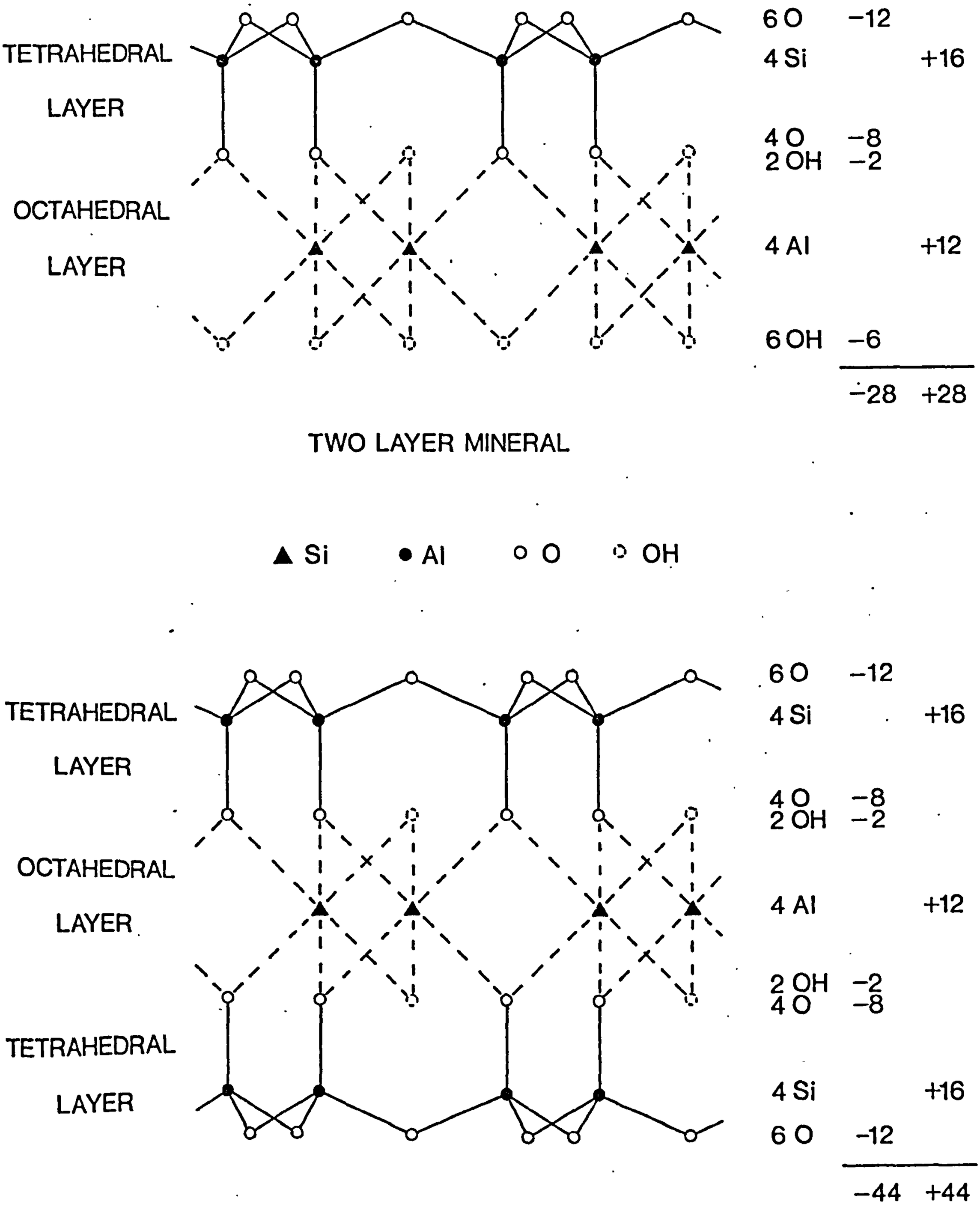


Fig.17 IDEALISED STRUCTURES OF MINERALS

is limited by the weak bonding forces between the units but the length and breadth are limited only by breaks in the lattices of the component layers and as a result the particles tend to have a plate like form. Hence the electrical charges due to lattice substitutions are mainly associated with the face areas rather than the much smaller edge areas.

2.2 The Clay Minerals

(i) Kaolinites

Kaolinites are two layer minerals with aluminium in the octahedral layer and the unshared oxygens of this layer replaced by hydroxyls. Very little substitution either for aluminium or silicon occurs and the units carry only a very slight charge. The main differences between the kaolinite minerals result from alterations in the stacking geometry rather than lattice substitutions.

It can be seen from Fig. 18 that if the two layer units of kaolinites are stacked one upon another an oxygen layer is adjacent to a hydroxyl layer thus:-

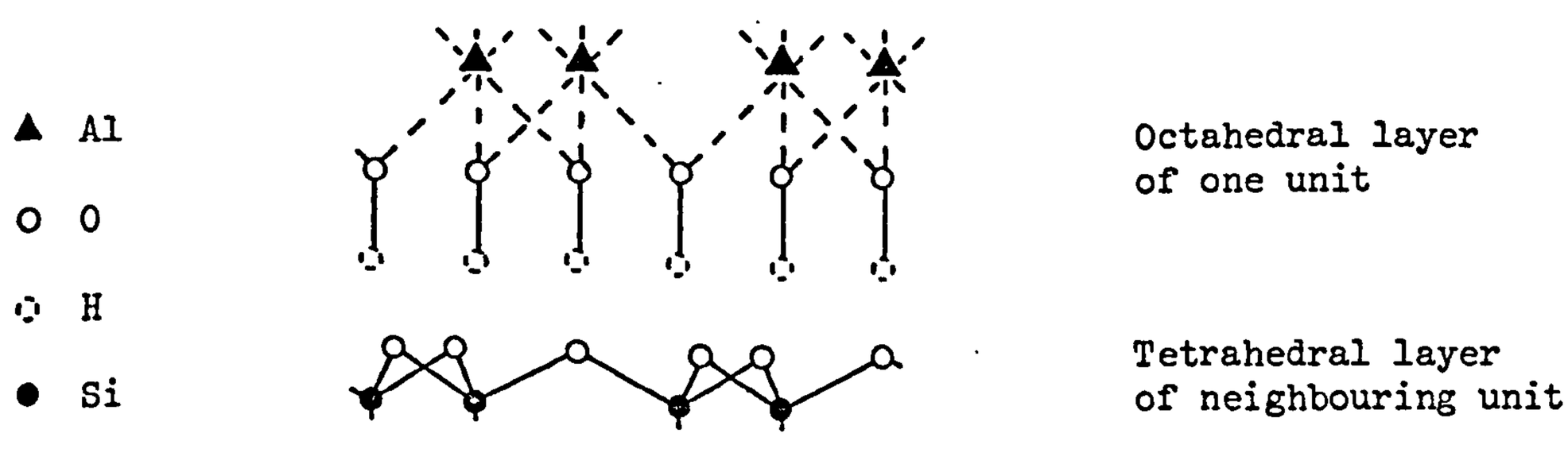


Figure 18

The hydrogen atoms of one unit are adjacent to the oxygen atoms of the neighbouring unit. This makes it possible for the hydrogen atoms to change from one unit to the other and they will in effect oscillate between the units. This oscillation produces a bonding known as hydrogen bonding, which though weaker than conventional chemical bonding holds the units sufficiently tight together to inhibit cleavage of the units. Thus it will not be easy to disperse kaolinite minerals into fine particles in water or for water to penetrate between the units. The lattice is said to be non-expanding in suspension as there is little tendency for water molecules to force the units apart.

Minerals in this group are kaolinite, dickite, and nacrite, serpentines are similar but have the aluminium replaced by magnesium.

(ii) Halloysite Minerals

Halloysites are very similar to kaolintes but in the stacking the units are displaced sideways so that the oxygen atoms of one unit are no longer adjacent to the hydroxyls of the neighbouring unit, and hydrogen bonding is very limited.

Halloysite exists in hydrated and dehydrated forms. In the hydrated form there is a single layer of water between the halloysite units. This water can be partially removed by heating to 65-70^{o}C and is totally and irreversibly removed at 400^{o}C. The hydrated form occurs as tubes made up from overlapping curved units. This curvature is probably induced by slight differences in the oxygen spacing in the octahedral and tetrahedral layers. The tubes tend to split or

unroll on dehydration as the bond between successive units is stronger without the interlayer water, and this stronger bond tends to pull the units flat. As in kaolinite there is very little lattice substitution.

(iii) Smectites (Montmorillonites)

Smectites are three layer minerals, that is they have two tetrahedral layers and a central octahedral layer. The units stack so that the oxygen atoms of successive layers are adjacent, but as there is no general replacement of oxygens by hydroxyls in the tetrahedral layer hydrogen bonding does not occur.

Unlike kaolinites and halloysites substitutions always occur in smectite lattices. Aluminium or more rarely phosphorus can replace silicon in the tetrahedral layer, and the aluminium of the octahedral layer can be replaced by iron, zinc, nickel, lithium or magnesium. Substitution in the tetrahedral layer is normally limited to about 15% of the available sites, but any degree of substitution can occur in the octahedral layer. Though normally either all the sites are filled (trioctahedral) or two thirds of the sites are filled (dioctahedral), the intermediate states do not seem to occur. The result of this and the substitutions is that the smectite lattice is always electrically unbalanced. Some internal compensation can occur if the imbalances on the octahedral and tetrahedral layers have differing signs or oxygen can be replaced by hydroxyl. The overall effect is to leave the lattice with a negative charge of about 0.66 electron units per unit cell. (The unit cell is the smallest portion of the lattice which if repeated will reproduce the structure.) This negative charge

is balanced by a cloud of positive metal atoms (cations) around the units. These are free to move and can readily be replaced by others provided that the overall suspension remains electrically neutral. This effect is known as cation exchange or more commonly base exchange.

The base exchange capacity of a clay is a measure of the electrical imbalance on the units. Thus kaolinite and halloysite will have very small base exchange capacities compared with smectites.

The negative charge of the smectite units produces a repulsive force between units and as the only attractions are from the weak Van der Waal's forces, smectite particles are readily dispersible in water to form very small flakes. Water can readily penetrate between the units to produce either a few molecular layers of interlayer water or complete separation of the units and thus smectites are said to have an expanding lattice.

The most common smectite is montmorillonite (Bentonite) which is based on an aluminium octahedral layer other smectites are nontronite which has iron in the octahedral layer and saponite which has magnesium.

(iv) Illites

Illites are three layer minerals similar to smectites except that the only substitution that occurs is aluminium for silicon in the tetrahedral layer and the resultant charge is always balanced by potassium ions between the units. The potassium ion is of just the right size to fit in the perforations between the oxygen layers of adjacent

units. The charge is about -1.2 to -1.5 per unit cell and as it is concentrated in the surface tetrahedral layers bonding between units by the potassium ion is possible. (This does not occur in smectites as the charge is mainly in the octahedral layer, at too great a distance from the surface) This bonding is strong enough to prevent penetration of water, thus illites have non expanding lattices and are difficult to disperse. Since the potassium is bound it will be difficult to exchange; thus illites exhibit very small base exchange values. However if the potassium is removed the inter-unit bonding will be broken and the degraded illite will behave very similarly to a smectite.

(v) Chlorites

These are complex minerals composed of aluminium based three layer minerals interleaved by units of octahedrally co-ordinated magnesium hydroxide. In the three layer units there is a net negative charge due to substitution of aluminium for silicon. The magnesium layer has a net positive charge from substitution of magnesium by aluminium. These charges neutralise each other and also link the structure together. Chlorites probably have a dispersibility similar to illites but they always occur mixed with other minerals and it is difficult to determine their properties.

Vermiculites are very similar to chlorites but the magnesium layer is not magnesium hydroxide $Mg(OH)_2$, but hydrated magnesium ions $Mg^{++}.nH_2O$ (n is not specified).

(vi) Mixed Layer Minerals

Many natural clay minerals contain units of different types. These units may be regularly repeated in the stacking as in chlorites or they may be randomly distributed. Mixtures of illite, smectite, chlorite and vermiculite units are common, but those involving kaolinites seem to be rare.

Mixed layer minerals are not difficult to disperse especially if they contain smectite units.

(vii) Allophanes

Allophanes are defined as those clay minerals which are amorphous to x-ray diffraction. This does not mean that they have no structural organisation but rather that the arrangement of tetrahedral and octahedral layers is either insufficiently regular, or the units too small to permit diffraction. Often the only evidence for allophanes is that the sum of the other constituents does not total 100%.

The properties of allophanes are very variable except that they generally have high natural water contents.

(viii) Attapulgite, Palygorskite and Sepiolite

These minerals have quite different structures to those previously described. Electron micrographs have shown that the particles are needle or lath shaped, very different from the flakes of most clay minerals.

Figure 19 shows the structure of attapulgite. The basic

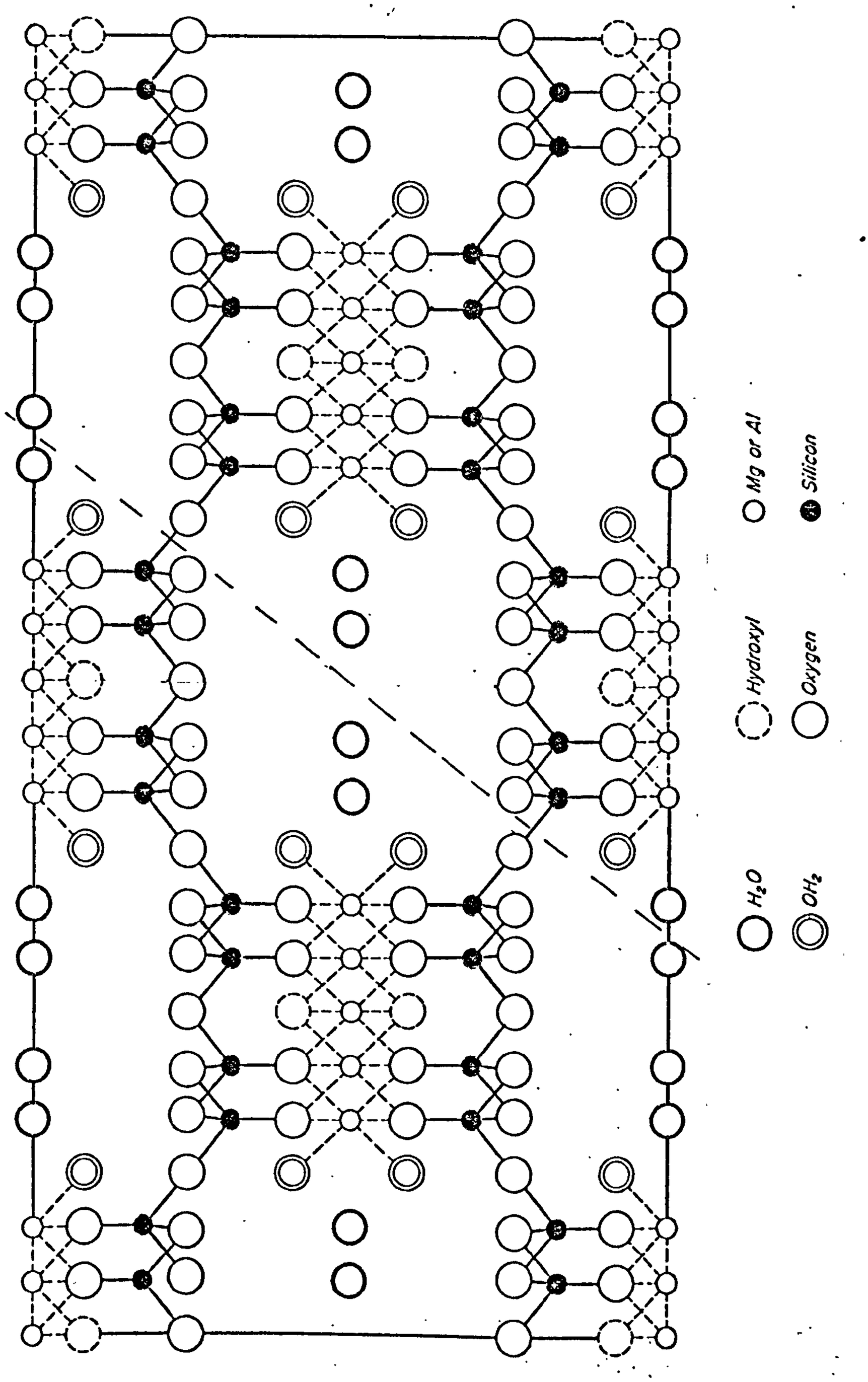


Fig.19 Attapulgite

building element is the tetrahedral silica layer but the vertices of the tetrahedra do not all point in the same direction. The tetrahedra are arranged in chains linked lengthwise so that the tetrahedra of alternate pairs of chains point in opposite directions, to produce a ribbed sheet. In attapulgite two of these ribbed sheets are linked together at the vertices by magnesium or aluminium. When dispersed in water cleavage occurs through the weak oxygen links as shown by the dotted line in Fig.19. The width of the laths in suspension depends on the amount of work (stirring) applied to break up the particles. Thus, for example, particle size analysis will only show the degree of dispersion and not the natural particle size. The structure has a number of channels running through it and these can be filled with water as shown in Fig.19 or by exchangeable cations.

Attapulgite and palygorskite have the same structure and it is probable that palygorskite is just a longer fibred version of attapulgite. Sepiolite has a very similar structure but the silica tetrahedra are inverted in groups of four.

2.3 The origin of clays

Many of the clay minerals have been synthesised in the laboratory, at moderate temperatures and pressures, from mixtures of silica and alumina with other metal oxides (Grim, 1962). Rocks containing silica and alumina can be similarly decomposed to yield clay minerals. In Nature this decomposition occurs in two distinct ways:

(i) Weathering

The type of clay mineral found in a given soil depends on the nature of the parent material, the climate, topography, vegetation, and time of weathering. However the following generalisations can be made:

Basic igneous rocks high in magnesium tend to produce smectite if because of poor drainage or low rainfall the magnesium is allowed to remain in the weathering zone after it is released by the breakdown of the parent material. If however the magnesium is quickly washed away kaolinite will be produced. Similarly acid igneous rocks high in potassium and magnesium will yield illite and smectite if there is no leaching and kaolinite if the potassium and magnesium are rapidly removed.

(ii) Hydrothermal alteration

The term hydrothermal is used to describe processes involving water vapour at pressures greater than one atmosphere and consequently temperatures greater than 100°C. Clay minerals formed by hydrothermal alteration are often found as an aureole around metalliferous deposits and the size of the alteration envelope may vary from a few feet to more than one hundred feet. Frequently the clays within this envelope have a distinct zonal relationship, for example kaolinite may occur near the centre with smectite and chlorite further out. In general the parent materials and leaching conditions which favour the formation of a particular clay mineral by weathering will yield the same mineral by hydrothermal alteration.

2.4 The Balance of Attractive and Repulsive forces on Clay Particles

a Face charges

If an electric field is applied to a clay suspension viewed under a microscope the clay particles will be seen to move towards the positive electrode. This effect is known as electrophoresis and demonstrates that the clay particles have a net negative charge. In the previous section it was shown that the source of this charge was substitutions in the clay lattice. To preserve overall electrical neutrality the clay particles are surrounded by a cloud of ions containing an excess of positive ions which are referred to as counter-ions. Figure 20 shows a typical particle and associated counter-ion cloud.

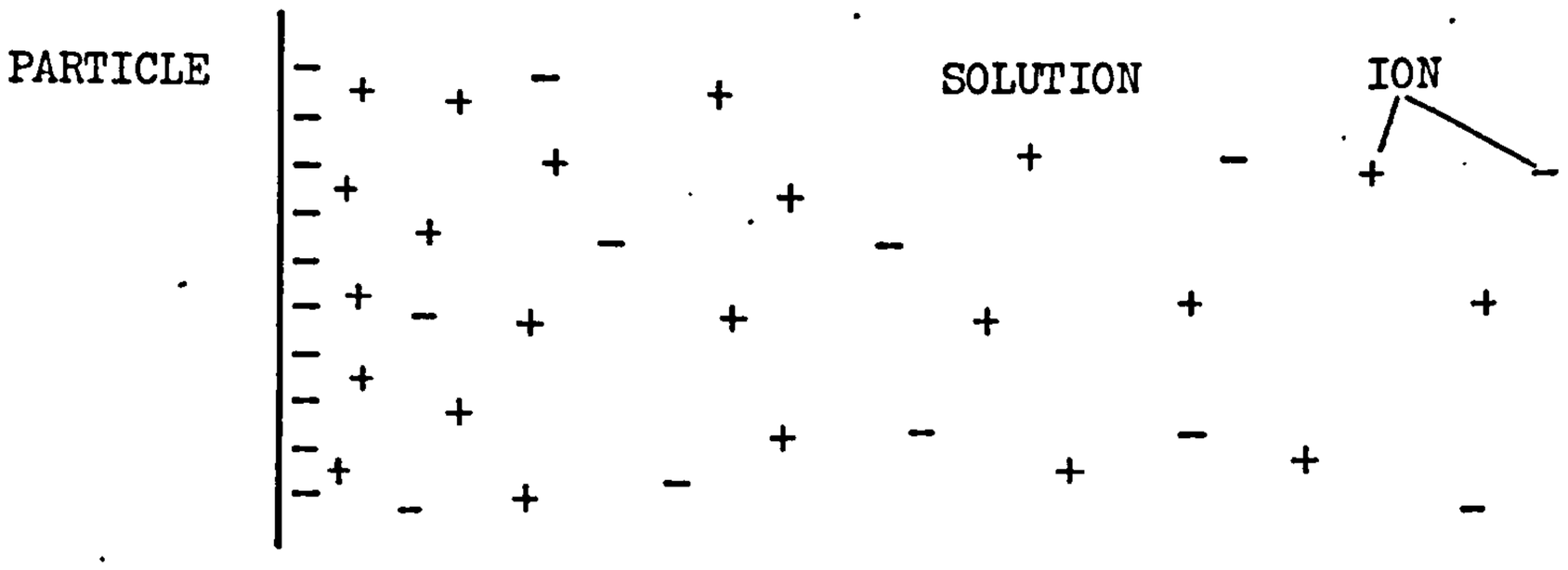


Figure 20

The charge in the counter-ion cloud is not concentrated at the surface of the particle but diffuses outwards so that the charge density decreases with distance from the particle.

If an electrolyte such as sodium chloride ($Na^{+}Cl^{-}$) is added to the solution the concentration of ions will increase, and it can be shown theoretically that this compresses the counter-ion cloud. The counter-ions move closer to the particle and the extent of the cloud is reduced. Higher valency ions have a much greater effect in compressing the cloud. Thus calcium (Ca^{++}) has a much greater compressive effect than sodium.

The particles in a clay suspension are continually in Brownian motion from collisions with water molecules and when two particles approach one another their counter-ion clouds will interact to produce repulsion. If the counter-ion clouds are compressed the particles will be able to approach more closely before this repulsion occurs. If this approach distance is sufficiently small the Van der Waal's forces may become dominant and link the particles together. Figure 21 shows the repulsive and attractive energies of two particles as a function of separation for three electrolyte concentrations. The Van der Waal's attractive energy is independent of electrolyte concentration.

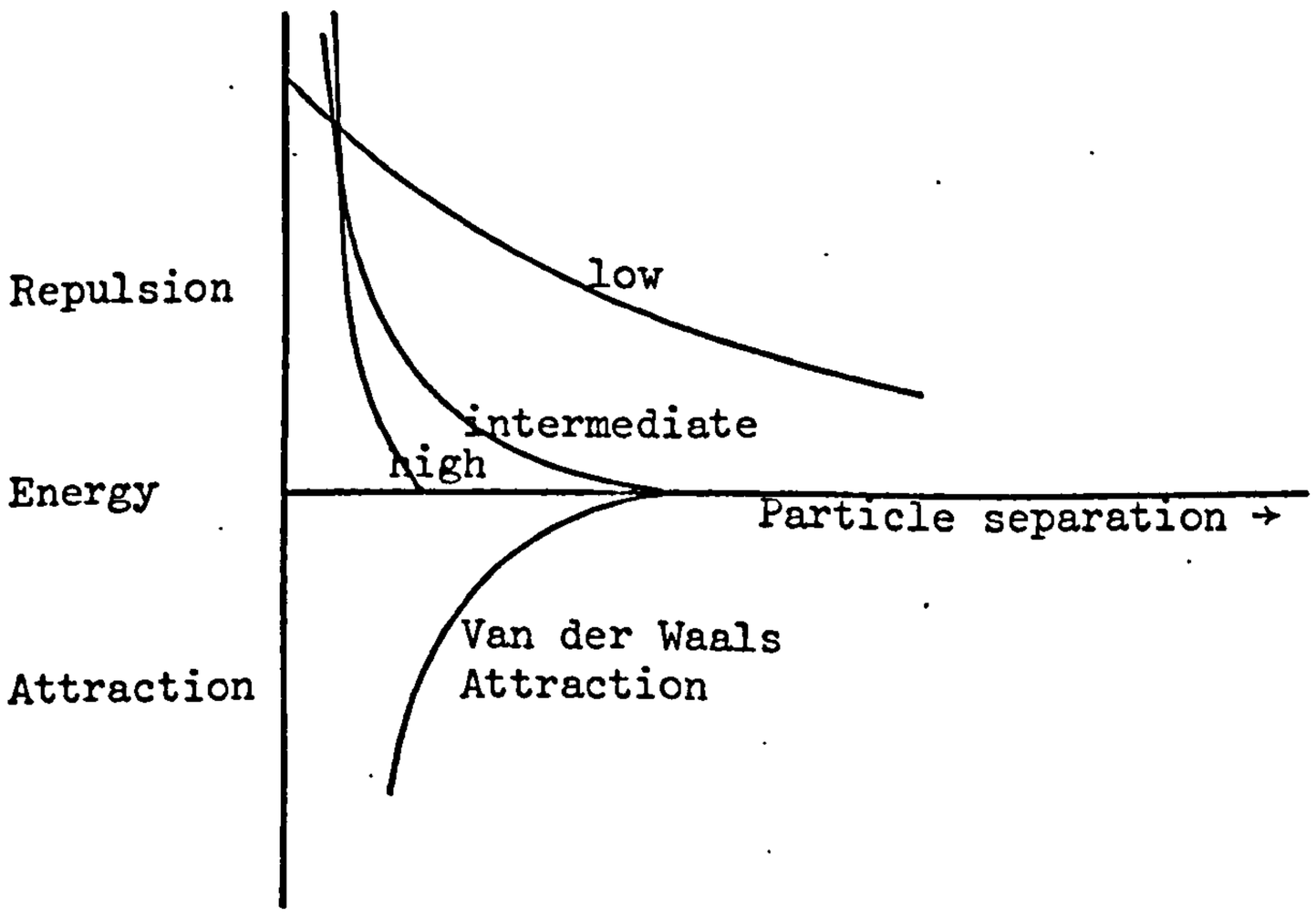


Figure 21 Repulsive and attractive energy as a function of particle separation at three electrolyte concentrations.

At very close approach distances two additional repulsive forces are observed. First as clay particles are surrounded by a layer of adsorbed water and this must be displaced and second as there is an electron repulsion which will prevent the penetration of one lattice by another.

Figure 22 shows the combined forces for the three electrolyte concentrations and it can be seen that the combined force varies from repulsion at low concentrations to attraction at high concentrations.

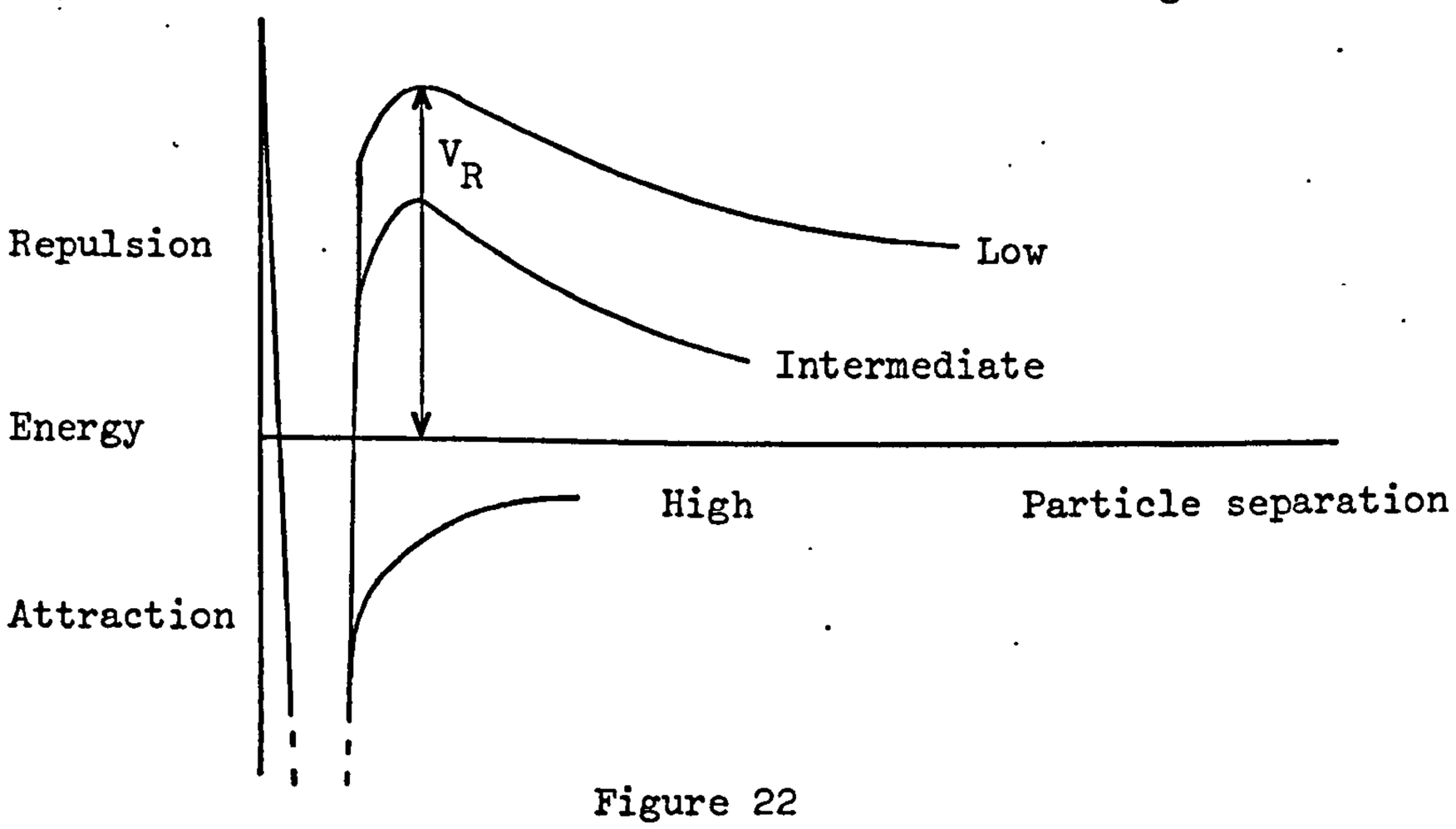


Figure 22

It should be noted that if at any electrolyte concentration the particles have sufficient energy from Brownian motion to approach each other close enough for the attractive forces to be dominant, they will be locked together and there will be a very high energy barrier against separation. Thus even without added electrolyte clays will link together at a rate dependent on the height of the energy barrier (V_R) compared with their kinetic energy from Brownian motion.

b Edge Charges

So far consideration has only been made of the face charges on clay particles, but at the edges the lattice will be broken and charges will result from the unsatisfied valency bonds.

The octahedral layer of a clay unit can break so as to leave an excess of either aluminium atoms or oxygens (or hydroxyls). Which of these is in excess depends on the acidity or alkalinity (pH) of the suspension. If an alkali such as sodium hydroxide (Na^+OH^-) is present there will be an excess of hydroxyl ions and any exposed aluminium will be covered by hydroxyl and the layer will have a negative charge. In acid solution there is a deficit of hydroxyl ions and any exposed hydroxyls will be removed to leave aluminium, and the layer will have a positive charge. At some pH not necessarily neutral the octahedral edges will have no charge. (The exposed aluminium or hydroxyl ions are referred to as potential determining ions).

In the tetrahedral layer the situation is more complex. Silica particles normally carry a negative edge charge but this is known to become positive in the presence of very small amounts of aluminium ion. Clay suspensions will normally contain small amounts of these ions from solution of the aluminium in the octahedral layer. Thus the tetrahedral sheet may well have a positive edge charge.

Thus under appropriate conditions the whole edge surface of clay particles will carry a positive charge but since the edge area is

very much less than the face area the particles will still have an overall negative charge even in acid solution when the edges have their highest positive charge. The edge charge, just like the face charge, will have a counter-ion cloud but of anions (negative ions). Thus clays will exhibit a small anion exchange capacity exactly analogous to base exchange capacity.

To summarise: clay particles have a negative face charge balanced by a cation cloud. This cation cloud prevents the close approach of particles. However the cloud and hence its range of effect can be compressed by adding electrolyte especially electrolytes with polyvalent cations. The edges of clay particles carry a positive charge and associated anion cloud, and this cloud can also be compressed by added electrolyte. The edge charge can be neutralised or reversed by adding alkali. Between clay particles there are also the Van der Waal's forces which are unaffected by electrolyte concentration or changing pH.

It is the balance between the electrical forces and the Van der Waal's forces that decides the physical properties of any clay suspension.

2.5 Particle Association

a Terminology

In a clay suspension three modes of particle association may occur: face-to-face (F-F), edge-to-face (E-F) and edge-to-edge (E-E).

F-F association or aggregation merely leads to thicker or possibly larger plates whereas E-F and E-E association, which are both referred to as flocculation, lead to three-dimensional voluminous "house of cards" structures. The particles are said to be dispersed if no F-F association occurs and deflocculated if there is no E-F or E-E association.

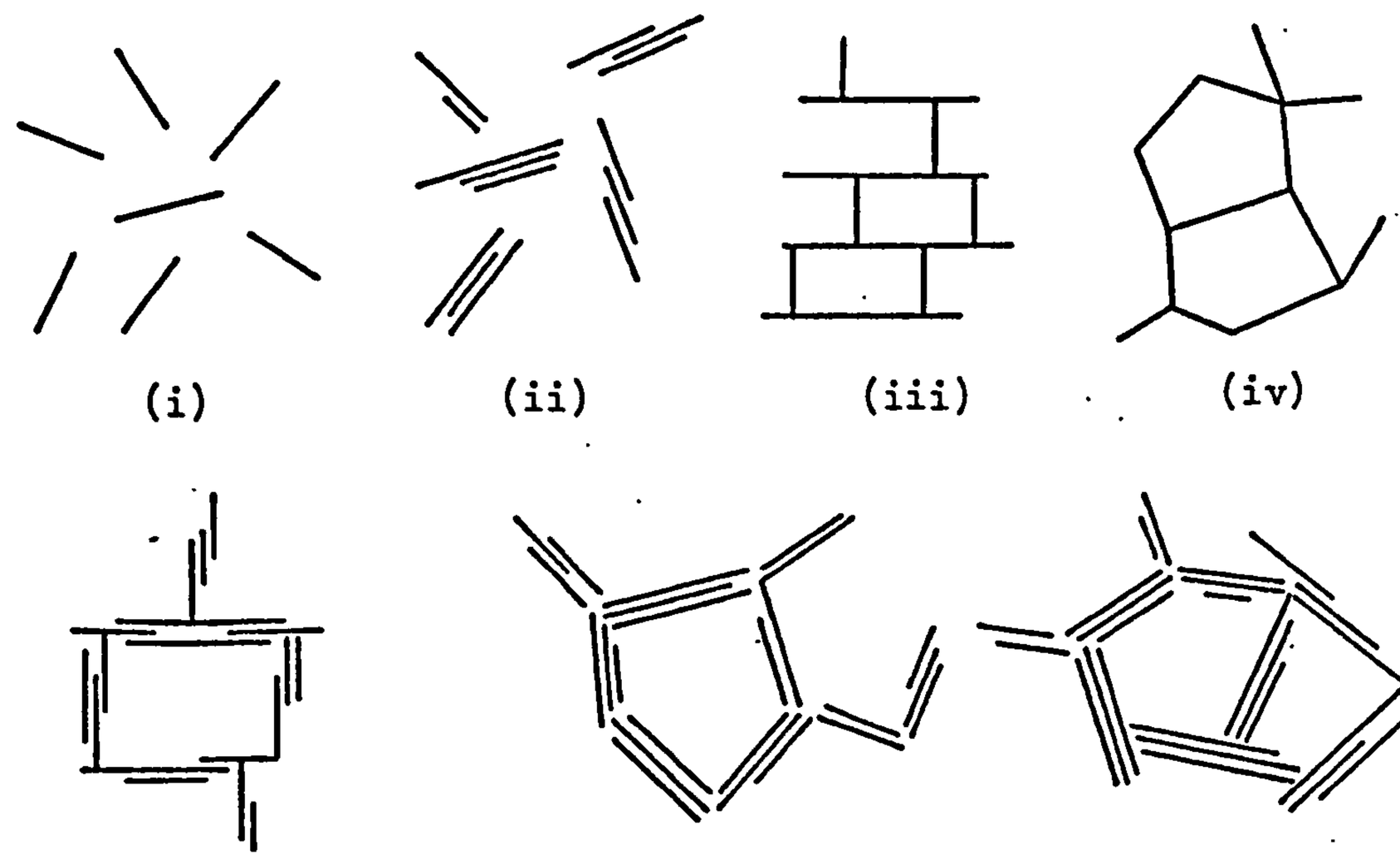


Figure 23

The various possible combinations of particle association are illustrated in Fig.23 and are defined as follows:-

(i) Dispersed and deflocculated

The particles are independent and there is no structure linking them.

(ii) Aggregated but deflocculated

The particles are linked by F-F association only, there is no structure

(iii) Edge-to-face flocculated but dispersed.

The particles are linked individually E-F to produce a house of cards structure, with no aggregation.

(iv) Edge-to-edge flocculated but dispersed.

(v) Edge-to-face flocculated and aggregated.

(vi) Edge-to-edge flocculated and aggregated.

(vii) Edge-to-edge and edge-to-face flocculated and aggregated.

b The effect on flow properties

The effect of particle association on flow properties is readily shown by the addition of electrolyte. In an electrolyte-free clay suspension both edge and face counter ion clouds will be well developed and Van der Waal association F-F and E-E will be prevented. However the opposite edge and face charges will cause E-F flocculation. The "house of cards" structure thus formed will give the suspension a high viscosity, and because of the continuous structure the suspension will exhibit a gel strength. That is to say, a certain minimum shear force will be needed to produce flow.

In the presence of very small amounts of electrolyte the counter-ion clouds are partially compressed. The small edge cloud is most effected and E-F association is prevented but face cloud is not sufficiently affected for F-F association by Van der Waal bonding to occur. The suspension is thus dispersed but not aggregated, hence the viscosity and gel strength will be low.

When more electrolyte is added the clouds will be sufficiently compressed for Van der Waal bonding, but this will be E-F rather than F-F as the attractive E-F and repulsive F-F forces will still have some

effect. The suspension is thus flocculated but not aggregated, and will have a high viscosity and gel strength.

At very high electrolyte concentrations the E-F and F-F electrical forces will cease to have any effect and F-F, E-F, E-E Van der Waal bonding will all occur. The suspension is thus flocculated but aggregated. The aggregation will lower the number of particles available to form a structure without strengthening the structural linkages. Thus the viscosity and gel strength will be lower than at intermediate electrolyte concentrations.

2.6 Chemical treating agents

Many special chemicals have been developed for the treatment of clay suspensions particularly drilling muds, and details of these can be found in Rogers (1948). A few of the more commonly used chemicals are listed here.

(i) Thinners

Thinners are used to lower viscosity and gel strength as follows:

Sodium polyphosphates

Tannins and quebracho

Lignins and Calcium and Chromium lignosulphonates.

All these are long chain polymers and with the exception of the polyphosphates and some mined lignins are obtained from wood extracts. Fig.24 shows a schematic diagram of such a polymer.

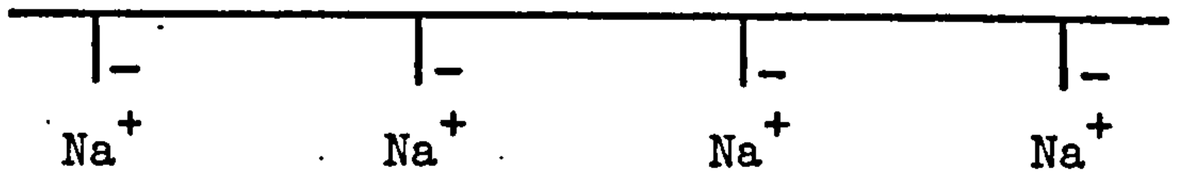


Figure 24

In solution the cations are released and the spacing of anion groups of the chain is such that they can link to the positive edge charges of the clay particles. Thus particle edge charges are neutralised and E-F flocculation will be inhibited. The cations released especially from the calcium or chromium (Cr^{+++}) salts may (depending on the electrolyte concentration) compress the face counter-ion clouds sufficiently to promote F-F aggregation. Thus flocculation will be inhibited and aggregation promoted, both of which reduce viscosity and gel strength.

Sodium polyphosphates should not be used if large quantities of calcium ions are present as they react to precipitate calcium polyphosphate and release sodium ions. The calcium polyphosphate can have no action on the clay edges, but the replacement of Ca^{++} by Na^{+} in the suspension may cause deflocculation at low electrolyte concentration but it will also cause dispersion (breaking up of aggregates). Deflocculation will lower gel strength and viscosity but dispersion by increasing the number of particles will cause increases and the final position may well be increased gel strengths and viscosities. Thus polyphosphates will act as dispersants rather than thinners for suspensions containing calcium ions. Sodium hexametaphosphate, a typical polyphosphate is recommended as a dispersing agent prior to particle size analysis in the standard specification for soil testing (British Standard Institution, 1967)

Tannins, lignins and quebracho have the advantage that they can thin at the high electrolyte concentrations which are often used in drilling to prevent dispersion of native clays from the formation. High pH (alkalinity) is however sometimes required which makes the drilling fluid dangerous to handle and tannin treated muds tend to solidify at the high temperatures often encountered in deep drill holes.

(ii) Flocculants

If the clay available to make a drilling mud or slurry cannot yield a satisfactory gel strength at an acceptable viscosity it is sometimes necessary to use a flocculant to improve the gel strength. This of course can be achieved by addition of electrolytes especially polyvalent electrolytes but the effect of these can be greatly enhanced if the clay is sensitised by the addition of an organic polyelectrolyte. These organic polyelectrolytes are very similar to the long chain thinners but the cation spacing is different so that they can bond to the edge of more than one clay particle.

Carboxymethyl cellulose (CMC) is the most commonly used flocculant and can be obtained commercially in several different chain lengths. If CMC is added to a dispersed suspension the particles will be too far apart for any interparticle bonding and the CMC chains will bond to individual particle edges just like the thinners and will as a result increase the tolerance of the suspension to electrolytes.

However if some electrolyte is added to the suspension prior to the CMC the interparticle distance may be sufficiently reduced for the chains to bond to several particles thus building a very effective gel structure. Only very small amounts of CMC are required, typically about 0·0004%. At higher concentrations the CMC will coat the clay particles and once again act as a thinner.

(iii) Fluid loss control Agents

It is essential both in drilling and slurry trenches that the clay suspensions should be able to form a filter cake of very low permeability on the walls of the hole. Starch, CMC, and polyacrylonitriles are commonly used to reduce losses through the filter cake. A preservative must be used with starch to prevent fermentation. Like CMC, starch and the polyacrylonitriles are also organic polymers and the method of control is probably by coating the clay particles with a slippery polymer layer so that they can pack more closely.

Thinners also act as fluid loss control agents, for by breaking gel structures they allow the clay particles to pack more closely together.

(iv) Lost circulation materials

Lost circulation materials are fibrous materials added to assist filter cake formation in gravel or sand beds and cracks in rock formations. Strictly these are not chemical treating agents but they are included here for completeness. A very wide range of materials is sold including mica, cellophane flakes, ground sugar cane

fibres, wood shavings and ground nutshells.

2.7 Clays used in drilling and slurry trenches

The flow properties of clay suspensions depend on the degree of dispersion of the clay particles. Smectites, particularly bentonite and attapulgite type minerals, can be finely dispersed and thus have good viscosity and gel properties. Kaolinite illite and halloysite tend to remain aggregated in suspension and at concentrations exhibiting satisfactory gel strengths the plastic viscosities are often very high. Chemical treatment is also difficult as the flow properties show little variation with electrolyte concentration as shown in Fig.25.

Bentonite is much the most commonly used clay both in drilling and slurry trenches except when high salt concentrations are likely to be encountered. Rogers (1948) quotes 10g/ℓ sodium chloride as the highest salt concentration at which bentonite can be used as the salt inhibits the dispersion of the dry bentonite and the viscosity and gel strength cannot develop and the filter loss becomes very high. It can however be used at higher salt concentrations if initially hydrated in salt free water; contact with salt will then flocculate the clay but this can be treated with thinners if necessary. The highest salt concentration normally encountered in slurry trenches is that due to contamination by sea water which has a concentration of about 3·5 g/ℓ and therefore does not greatly affect a bentonite slurry. Chemical treatment may however be necessary when digging in chalk (calcium carbonate) or gypsum (calcium sulphate) areas as the calcium ions released from the slight solubility of these materials will have a

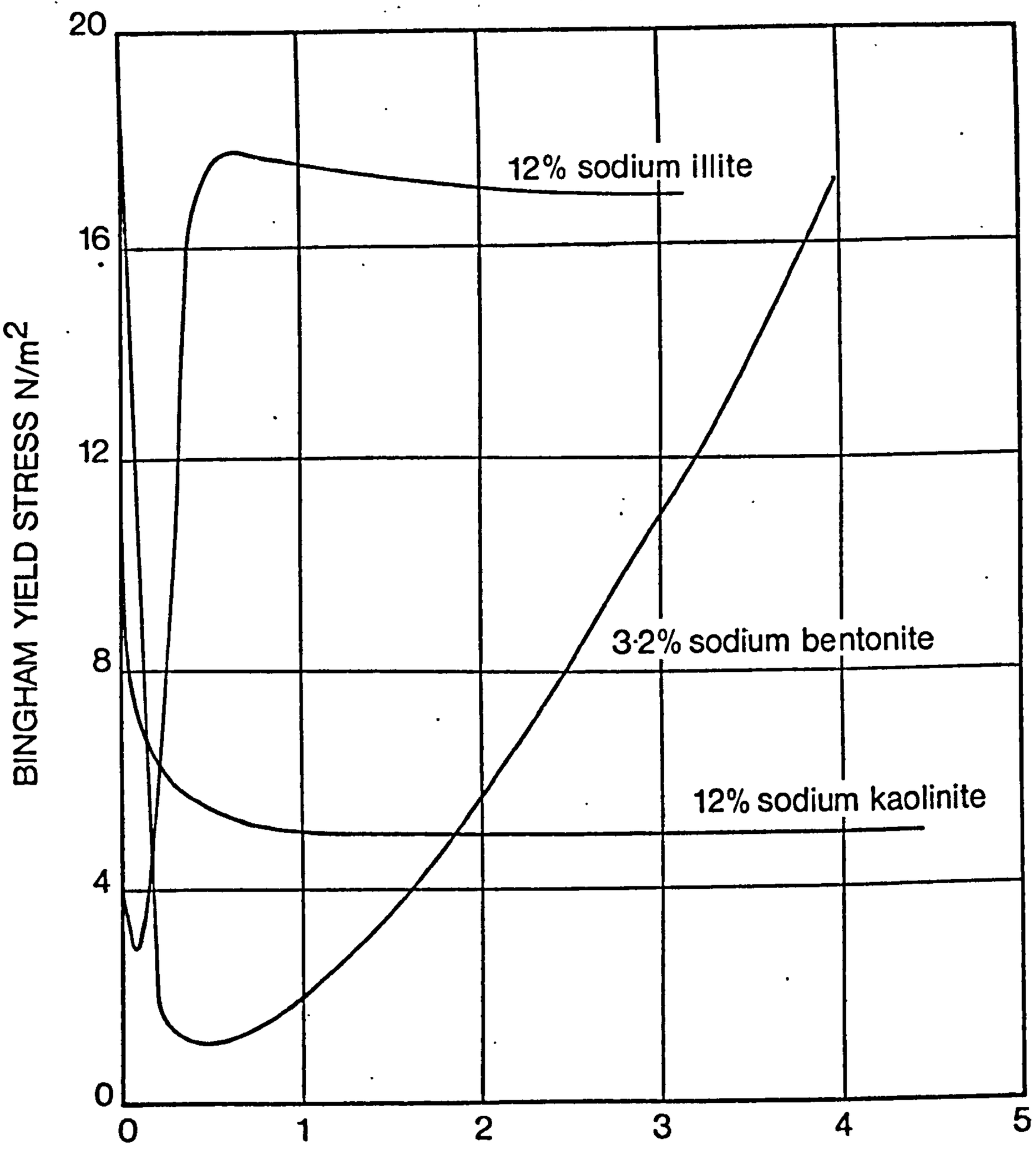


Fig. 25 BINGHAM YIELD STRESS OF CLAY SUSPENSIONS AS A FUNCTION OF SALT CONCENTRATION

much greater flocculating effect than say sea water which contains mainly sodium ions.

For high salt concentrations such as those encountered in drilling salt sections attapulgite clays are used as the gel strength in these clays probably results from the tangling of the laths and is thus independent of salt concentration. Attapulgites do however have to be mixed for considerably longer than bentonite as the laths do not readily disperse. They also have high filtrate losses.

2.8 Analysis of attapulgite and bentonite

Attapulgite is mined almost exclusively in a small region of the United States of America and is not often encountered elsewhere. Analysis is normally made by x-ray diffraction and electron diffraction techniques. Bentonite however occurs in many parts of the world and it is sometimes necessary to test deposits for purity and quality. This can also be done by diffraction techniques but they are laborious and some rather simpler laboratory tests are available.

The liquid limit test provides a simple indication of the presence of bentonite. If the liquid limit of a clay is over 100% bentonite is probably present, if it is over 250% the bentonite is probably in the sodium form though a lower value does not exclude this. Additionally if the clay readily disperses in water to produce viscosity and gel strengths similar to those shown in Fig.4 a sodium bentonite is present.

Jones (1964) developed a chemical test by which the percentage of bentonite in a clay can be estimated. The basis of this test is that bentonite has a much higher cation exchange capacity than the other common clay minerals and this capacity can be measured by the adsorption of a coloured cation such as that in the dye methylene blue. To make the test a known quantity of a dilute suspension of the clay is placed in a conical flask and treated with sulphuric acid to convert it to the hydrogen form (if the clay is not first converted to the hydrogen form the exchange for methylene blue may be rather slow as cations other than hydrogen may not be readily displaced). Methylene blue solution is then run in from a burette a little at a time.

After each addition the flask is shaken and a drop of liquid removed and touched on a filter paper to give a spot of coloured solids and initially a colourless liquid spreading outwards. When the end point at which the clay can adsorb no more dye is reached the spreading liquid becomes coloured. The quantity of methylene blue required to reach this end point gives an indication of the cation exchange capacity of the clay. Table II shows typical methylene blue adsorption capacities of the common clay minerals.

Table II

Clay mineral	Adsorption capacity grams dye/gram clay
Bentonite	0.27
Kaolinite	<0.05
Illite	<0.05
Attapulgite	0.12

For a mixed clay the contribution from illite and kaolinite can be ignored and as attapulgite is unlikely to be present the percentage of bentonite can be calculated from the weight of dye adsorbed by a known weight of the clay.

If the liquid limit and viscometric data do not indicate a sodium bentonite it is often necessary to analyse the exchangeable cations as the clay must normally be converted to the sodium form prior to use. For analysis the cations must be leached from the clay. Ammonium salts are often used for this, as the ammonium ion (NH_4^+) can displace the common cations, sodium, potassium, calcium and magnesium from most minerals, probably because its larger size results in a better attachment to the clay. After the leaching process the ammonium salt is analysed for additional cations; this is a complex process as the concentrations are low but for sodium, potassium and calcium a quick qualitative indication of the salts present can be obtained from the colour the solution imparts to a flame.

CHAPTER THREE

THE SLURRY TRENCH CUT-OFF FOR HYDRAULIC STRUCTURES

3.1 Introduction

(a) Construction
(b) Specifications for backfill material

3.2 Experimental work

(a) Initial test programme
(b) Backfill materials
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(e) Calibration of the hydraulic system
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(g) Water content measurements

3.5 Test results

(a) Permeability tests
(b) Consistency and water content tests

THE SLURRY TRENCH CUT-OFF FOR HYDRAULIC STRUCTURES

3.1 Introduction

a. Construction

The slurry trench method of construction consists of excavating a vertical trench which is kept filled with slurry and then backfilling this trench with selected material to produce a cut-off. The excavation is normally done with a specially designed backhoe (e.g. Sadleir and Dominioni, 1963) or dragline and as the spoil is removed clay slurry is pumped in to maintain the slurry level. When a length equal to about twice the depth of the trench has been excavated backfilling commences, though in stable ground much longer unfilled lengths may be employed. If the backfill is very fluid after placement it may again be necessary to leave long unfilled sections or to insert a barrier across the trench to prevent excavation of the placed material. The backfill procedure must be carefully controlled, for example if the fill is allowed to drop vertically through the slurry, segregation may occur so that the coarse fraction collects at the bottom of the trench and the fines remain in suspension. Also the falling material may trap pockets of slurry. To avoid these problems the first section of fill is often placed using a clamshell bucket which is lowered to the submerged fill surface before it is opened. Once the fill placed by this method has reached the slurry surface further material is dumped in a low pile at the top of the trench and pushed in with a bulldozer. The material should then slide as a thin sheet

down the sloping surface of the previously placed fill, thus avoiding the problems of vertical placement. For very cohesive fills which will not slide the clamshell bucket may be used throughout. If concrete is to be placed as for the construction of underground walls a tremie pipe is normally used and this can be used for other materials if segregation is a serious problem.

The backfill material must be capable of creating a cut-off as placed without further compaction as attempts to compact the submerged fill may well produce a stirring action which will disperse the material in the slurry. Surface rolling can be used at the top of the trench but this will produce a localised effect due to arching in the fill.

b. Specifications for backfill material

Kotowicz (1967) quotes a design permeability of 2×10^{-6}cm/s for a two metre wide cut-off and this would seem a typical design requirement though in situ permeabilities as low as 5×10^{-9}cm/s have been quoted (La Russo, 1963). To produce a permeability of the order of 10^{-6}cm/s the backfill must have a significant clay fraction. Ideally the grading should be such that there is only just sufficient fine material to fill the voids between the coarser particles. This will give a fill of low compressibility which is particularly important if there is to be further construction over the trench, as for example with the cut-off under a dam. A fill with this minimum fines content will however be very vulnerable to segregation and extra fines are normally allowed. Backfill within the following gradation limits has been used on a large number of projects.

Screen size B.S.S.	Percent Passing by weight
3 inch	80-100
¾ inch	40-100
¼ inch	30-70
No.25	20-50
No.200	10-25

Table III

The backfill may be placed dry if it is sufficiently friable or alternatively the water content may be adjusted so that it has sufficient slump to flow smoothly into the trench.

3.2 Experimental work

a. Initial test programme

In the first section of this research two aspects of the backfill procedure were investigated.

1. What permeability can be achieved by bulldozing dry material into a slurry trench and how does this change with time?

2. Does this method of placement lead to pockets of slurry being trapped in the fill and if so are these significant?

The materials used in the investigation were those used to construct a slurry trench cut-off in the Dead Sea Dykes. These Dykes were built to divide part of the South Basin of the Dead Sea into a series of evaporating pans for salt extraction. They were

built by dumping and bulldozing a sand/gravel mixture into the Sea which was about 4-5 metres deep in this region and when completed they stood about 3 metres above Sea level. A two metre wide slurry trench was then excavated along the centre line and carried down to a clay layer about five metres below the Sea bed (Fig.26).

b. Backfill materials

There were two types of backfill available near the site and both of these were used as borrowed:-

(i) Dry Clay

A light grey green very silty clay with a natural water content of about 13%:Atterberg Tests gave the following average values:-

Liquid Limit 35%

Plastic Limit 17%

Typical grading curves for wet and dry analysis are shown in Fig.27 and it can be seen that the lumps broke down very considerably when dispersed in water.

(ii) Asphaltic Clay

A grey brown friable material containing occasional hard lumps. The average water content was 14% and the Atterberg limits were:

Liquid limit 50%

Plastic limit 26%

Fig. 28 shows a typical grading curve for wet and dry analyses.

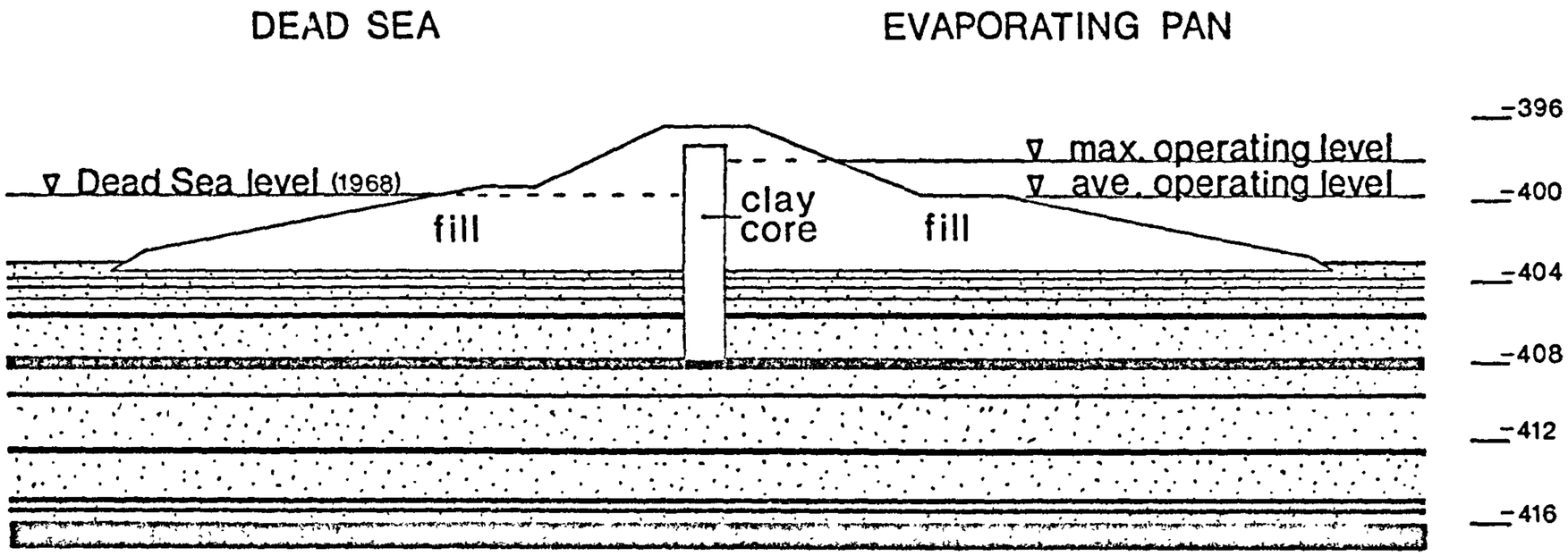
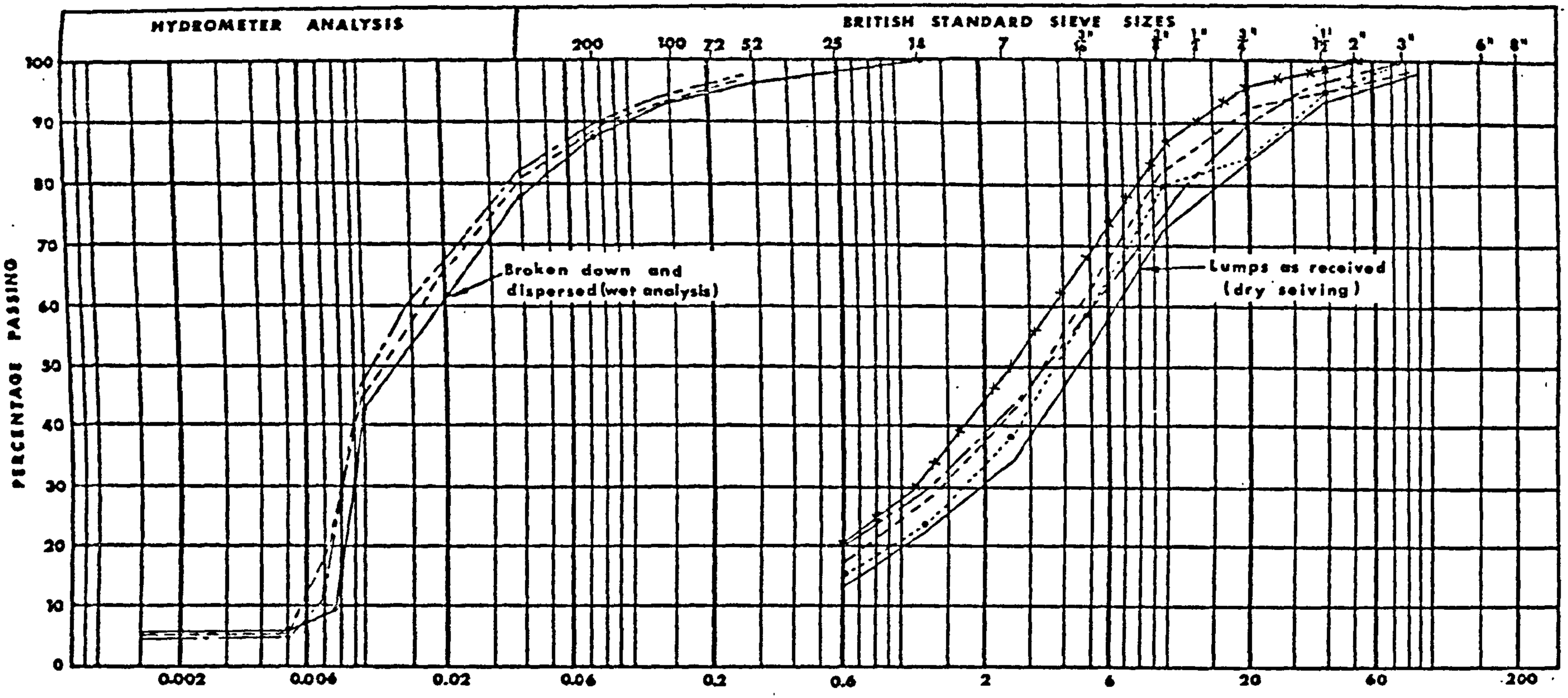


Fig. 26 TYPICAL DYKE SECTION



Drum No.	Sample No.	Depth feet	Graph Symbol	Visual Description of Soil	Wet / Dry Sieving
112	-	-	———	Typical samples of Dry Clay	Wet & Dry
118	-	-	– – – –		Wet & Dry
126	-	-	——–		Wet & Dry
143	-	-	.....		Dry
144	-	-	×–×–×–×		Dry

PRETREATMENT DETAILS (Hydrometer Analysis):

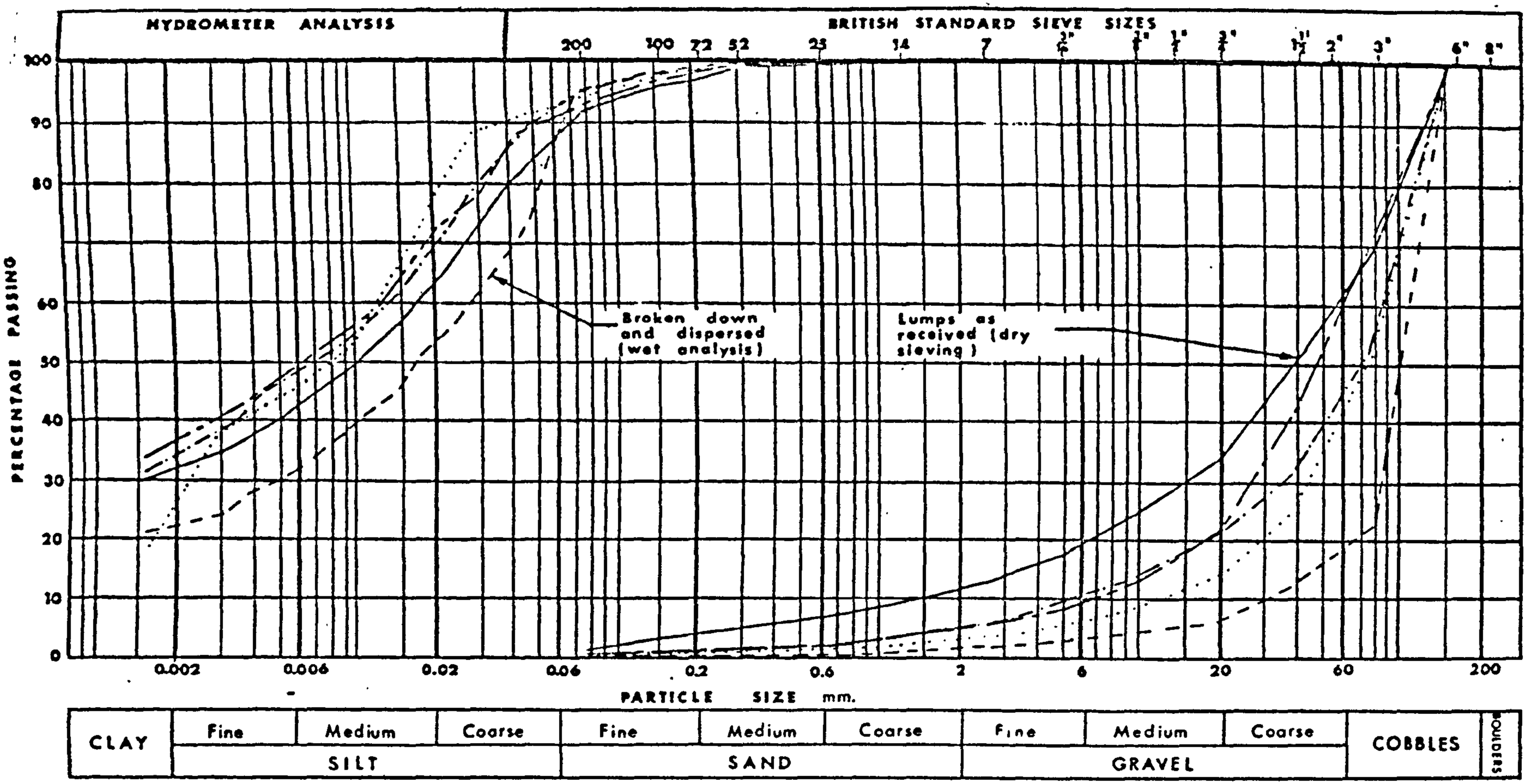
200cc. Hydrogen Peroxide 20 vols.

100cc. Sodium Hexametaphosphate 4%

DRY CLAY

PARTICLE SIZE DISTRIBUTION

Fig.27



Boring No.	Sample No.	Depth feet	Graph Symbol	Visual Description of Soil	Wet / Dry Sieving
			———		
			– – – –		
			——— –	Typical samples of Asphaltic Clay	Wet & Dry
			.....		
			—.—.—		

PRETREATMENT DETAILS (Hydrometer Analysis):

300cc. Hydrogen Peroxide 20 vols.

100cc. Sodium Hexametaphosphate 4%

ASPHALTIC CLAY

PARTICLE SIZE DISTRIBUTION

Fig. 28

The bituminous nature of this clay made it difficult to wet and when placed in water appreciable quantities of air were often trapped with it.

c. <u>The Slurry</u>

Dead Sea water contains about 119g/ℓ of sodium chloride and 142g/ℓ of magnesium chloride.* Dry bentonite could not be dispersed in such a salt solution and a suspension of bentonite in fresh water would immediately aggregate on contact with the salt. Attapulgite was therefore chosen as the basis of the slurry as it is not affected by salt concentration and does not require initial hydration in fresh water. The slurry was therefore made up with Dead Sea water with a consequent saving of fresh water and an improvement in the slurry density (Dead Sea water has a density of 1·23g/cm^3). Native clays were added to the attapulgite suspension and the slurries produced were named after the clay type. Their compositions were as follows:-

Dry Clay and Asphaltic Clay slurries

	Weight in grams for one litre of slurry
Attapulgite	39·9
Dead Sea water	1070·4
Dry or Asphaltic clay (<4·76mm)	169·7
Fine/medium sand	79·7

For the laboratory tests the components were mixed with a propeller type stirrer for ninety minutes, the sand being added to simulate trench

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* A physical and chemical analysis of Dead Sea water is given in Appendix I.

conditions after excavation.

Plastic Clay Slurry

The Plastic Clay, a heavy cohesive material dredged from the bottom of the Dead Sea was mixed with Dead Sea water to produce a raw slurry which was conditioned as follows for use in the trench:

	Weight in grams for one litre of slurry
Attapulgite	7·8
Dead Sea water	341·4
Raw Slurry (Density 1·62g/cm^3)	1157·5
Lime (Calcium hydroxide)	9·7

Again in the laboratory these components were mixed for four hours and then 3·6g of magnesium oxide were added and mixing continued for a further thirty minutes.

d. Physical properties of the Slurries

Table IV shows the physical properties of these slurries.

Slurry Type	Density g/cm^3	Plastic Viscosity Centipoise	Initial Gel Strength lb/100sq.ft	10 Minute Gel Strength lb/100sq.ft
Dry Clay	1.36	16	14	18
Asphaltic Clay	1.36	17	13	13
Plastic Clay	1.52	15	20	28

Table IV

The Plastic Clay slurry contained a much lower concentration of attapulgite than the other two. This was offset by a longer mixing time to disperse more of the attapulgite and Plastic Clay and also by the addition of magnesium oxide which may react with the magnesium chloride in the Dead Sea water to produce Soret's cement by the following reaction:-

$$\underset{\text{magnesium oxide}}{3MgO} + \underset{\text{magnesium chloride}}{MgCl_2} \rightarrow \underset{\text{Sorets cement}}{3MgO.MgCl_2.11H_2O}$$

This cement may improve the long term gel strength of the slurry. The lime may also form a cement as it reacts with magnesium chloride to produce magnesium hydroxide:-

$$\underset{\text{lime}}{Ca(OH)_2} + \underset{\text{magnesium chloride}}{MgCl_2} \rightarrow \underset{\substack{\text{magnesium hydroxide} \\ \text{(solid)}}}{Mg(OH)_2} + \underset{\substack{\text{calcium chloride} \\ \text{(soluble)}}}{CaCl_2}$$

Magnesium hydroxide is chemically very similar to magnesium oxide and this may form the cement.

e. <u>Soil/Slurry Combinations</u>

Three different soil/slurry combinations were used on site namely:-

Dry Clay in Dry Clay slurry
Dry Clay in Plastic Clay slurry
Asphaltic Clay in Asphaltic Clay slurry.

These materials were all tested in the laboratory and in addition, the Dry and Asphaltic Clays were tested in Dead Sea water alone without

slurry. Thus five different soil pore-fluid combinations were tested, and for each of these the changes in permeability and homogeneity with time were investigated.

The basic apparatus for the tests was a permeameter in which the sample could be subjected to a known constant overburden pressure and from which the sample could be removed intact at the end of the test so that a large number of consistency and water content measurements could be made to assess the homogeneity. The tests were continued for up to one year to investigate time effects.

3.3 The apparatus

a. The permeameter

As there was no suitable commercially available permeameter for clays the apparatus shown in Fig.29 was specially designed for this work.

The permeameter is made from a cylindrical Perspex cell in which a close fitting rubber piston can move. The sample is placed between two porous stones and the piston which is sealed against the cell wall by the "O" ring assembly can be hydraulically loaded to the desired overburden pressure. The top of the cell is sealed by a rubber gasket and a brass plate which is held down by six tie bars to a similar plate across the base of the cell. Perspex was used for the cell to avoid corrosion problems with the Dead Sea materials and also so that the sample under test would be visible. The walls of the cell were 25mm thick to reduce strain under pressure.

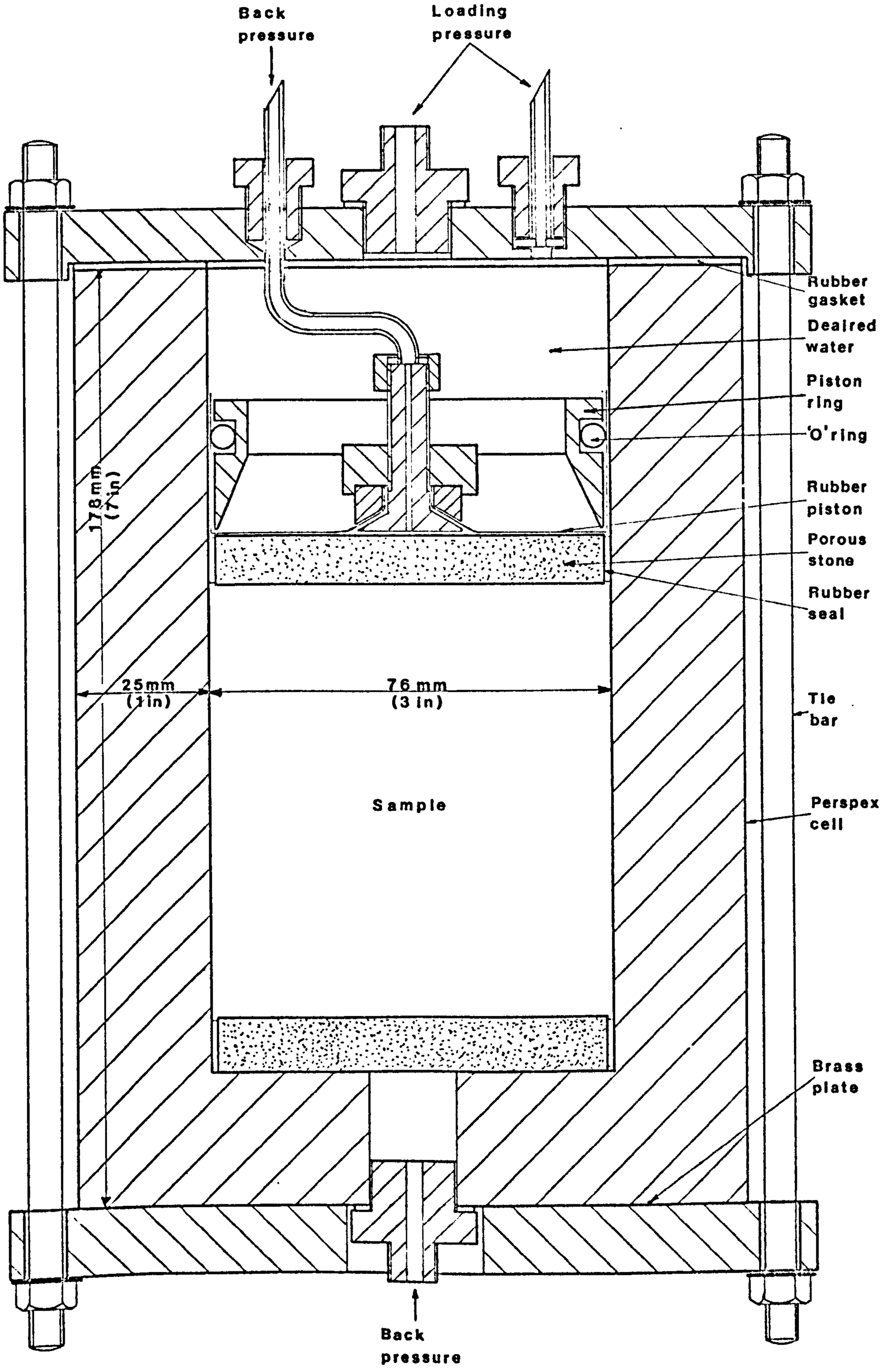
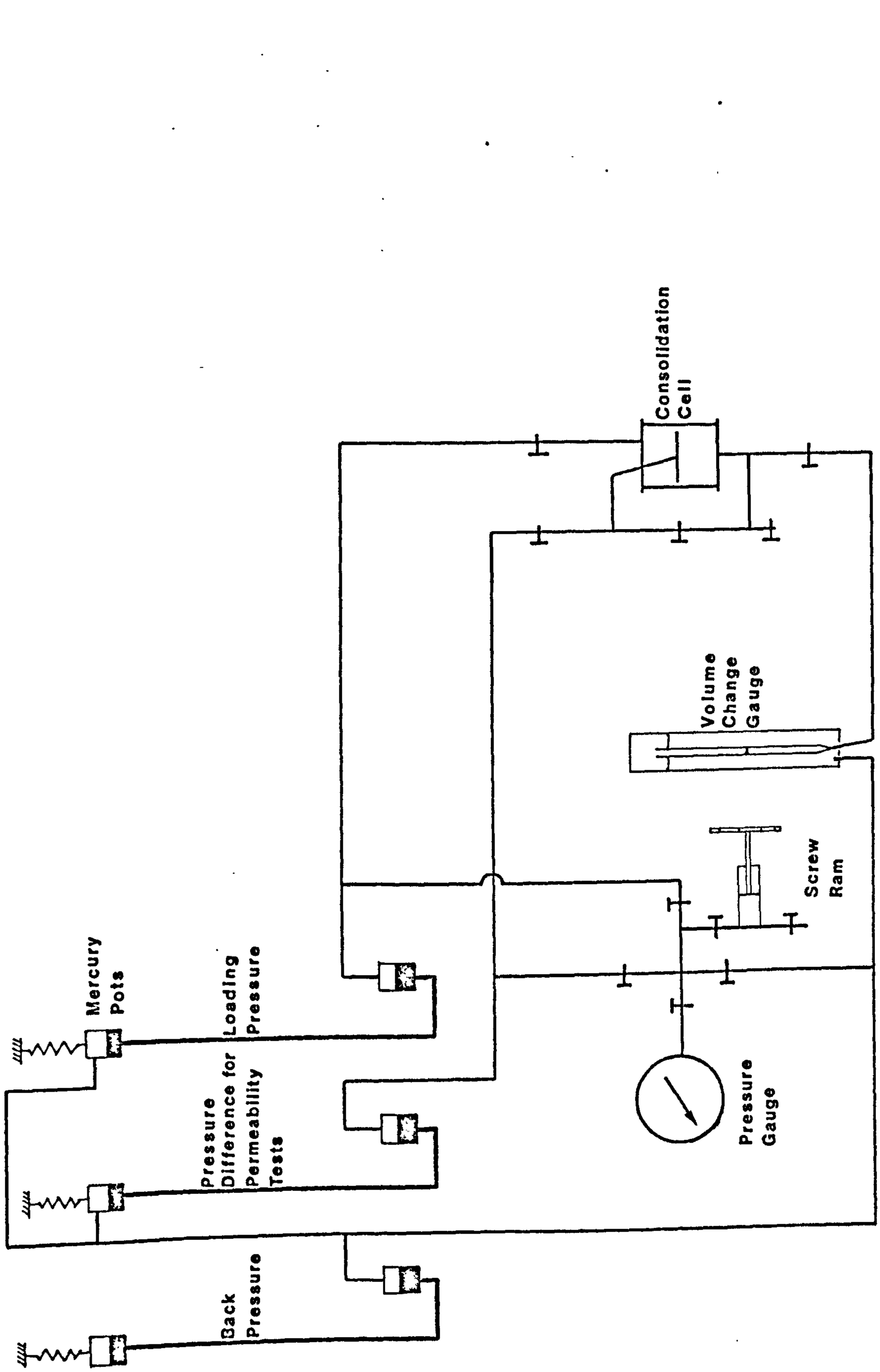


FIG. 29 THE PERMEAMETER CELL

Before assembling the ten cells that were required for the full test programme one cell was made up as a prototype. The cell was connected to a standard laboratory pressure control panel as shown in Fig.30 and the flow from the sample monitored with a commercial kerosene/water volume change gauge (Fig.31). A sample of Harmondsworth brickearth, a lean orange brown silty clay was tested in the cell with deaired distilled water as a pore fluid. Brickearth was chosen for this test as it was readily available in the laboratory and had been used in previous research and was thus well characterised. A back pressure of 30 psi (20·7kN/m^2) was used and the sample consolidated under overburden pressures of 4, 8, 16 psi (2·7, 5·5, 11·0 kN/m^2) in excess of the back pressure. For each pressure increment the rate of consolidation was calculated from observations of the meniscus in the volume gauge. When consolidation was complete (at 16 psi) the burette in the volume gauge was changed for a calibrated capillary tube so that much smaller volume changes could be measured. A pressure difference of 1 psi was then applied across the sample (equivalent to a gradient of about 10) and the permeability obtained from the flow rate as in a constant head permeability test. The slight change in differential pressure (about 2cm of water) due to the vertical movement of the kerosene/water interface during the test was ignored in comparison with the applied pressure difference (about 70cm of water). The measured permeability at the 16 psi effective stress was $1{\cdot}1 \times 10^{-8}$ cm/s. It was also possible to calculate the permeability indirectly from the Terzaghi one-dimensional consolidation theory using the relationship:-



HYDRAULIC DIAGRAM FOR THE PROTOTYPE PERMEABILITY CELL

FIG.30

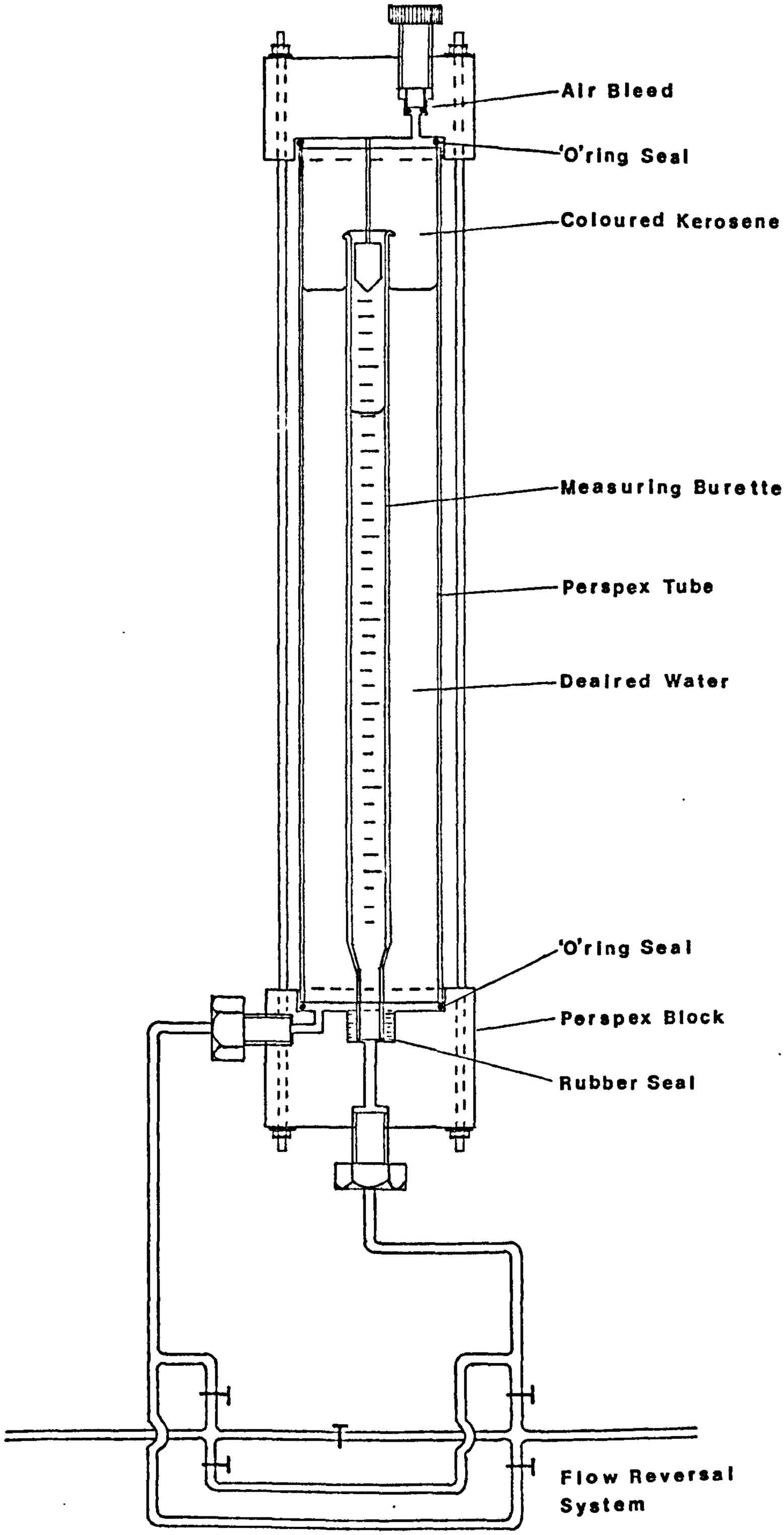


FIG.31 DIAGRAM OF THE KEROSENE / WATER VOLUME CHANGE GAUGE

$$k = c_v \cdot m_v \cdot \gamma_w \qquad \text{(Terzaghi, 1943)}$$

where k is the permeability

c_v is the coefficient of consolidation

m_v is the coefficient of compressibility

γ_w is the density of the pore fluid.

This led to a calculated permeability for the effective stress range 8-16 psi of $2{\cdot}2 \times 10^{-8}$cm/s. For this stress range calculated results were also available from two other laboratory oedometers:-

Direct loading machine	$8{\cdot}5 \times 10^{-9}$cm/s
Hydraulic loading machine	$1{\cdot}5 \times 10^{-8}$cm/s

From these results it was concluded that the cell worked satisfactorily and that there was no leakage round the edge of the sample as was feared could occur.

b. <u>The hydraulic system</u>

For the tests on Dead Sea materials a maximum effective stress of 8psi was used to simulate typical trench conditions and as each sample was to be subjected to several permeability tests it was decided to restrict the gradient used in these tests to about 1 - 2 to avoid significant swelling or consolidation of the sample each time the gradient was applied. A gradient of 1 corresponds to a head difference across the cell of 7-8cm of water. It was thus apparent that the commercial volume change gauge used for the prototype tests

could not be used for tests at this gradient as the pressure change it produced during a test would now be very significant. The volume gauge shown in Fig.32 was therefore developed for tests at low gradients.

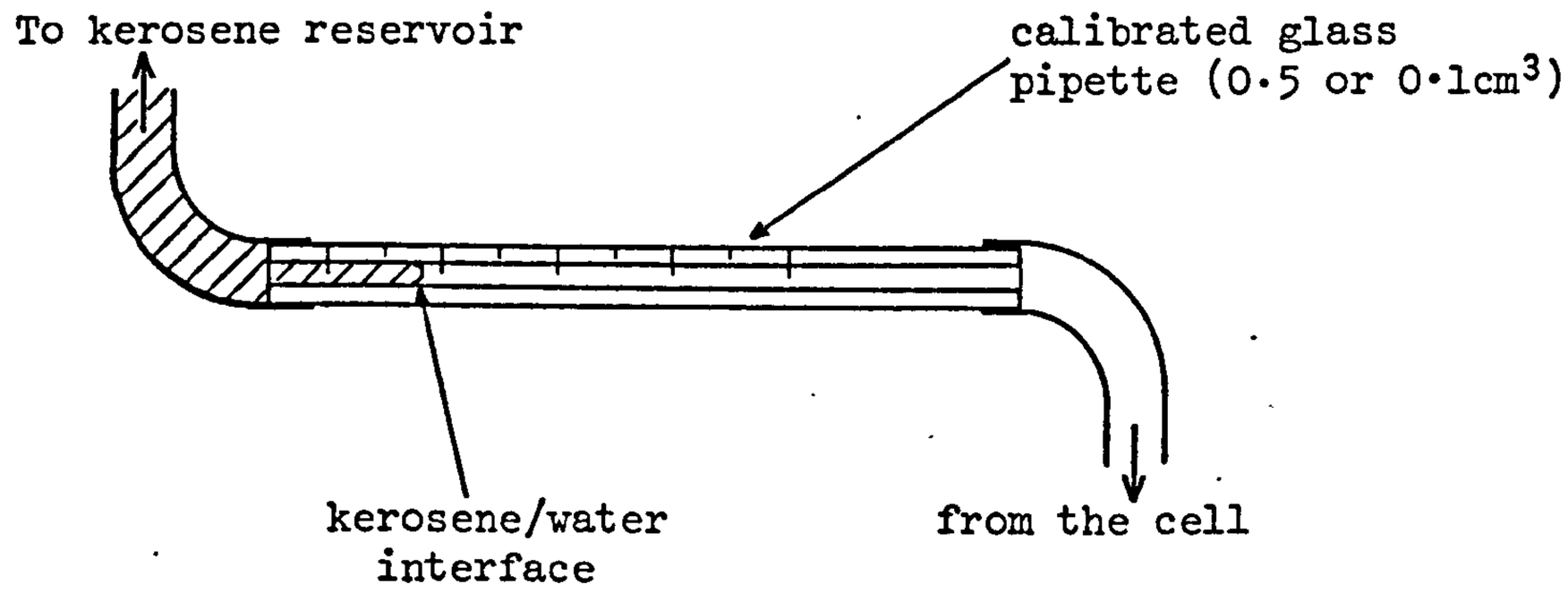


Figure 32

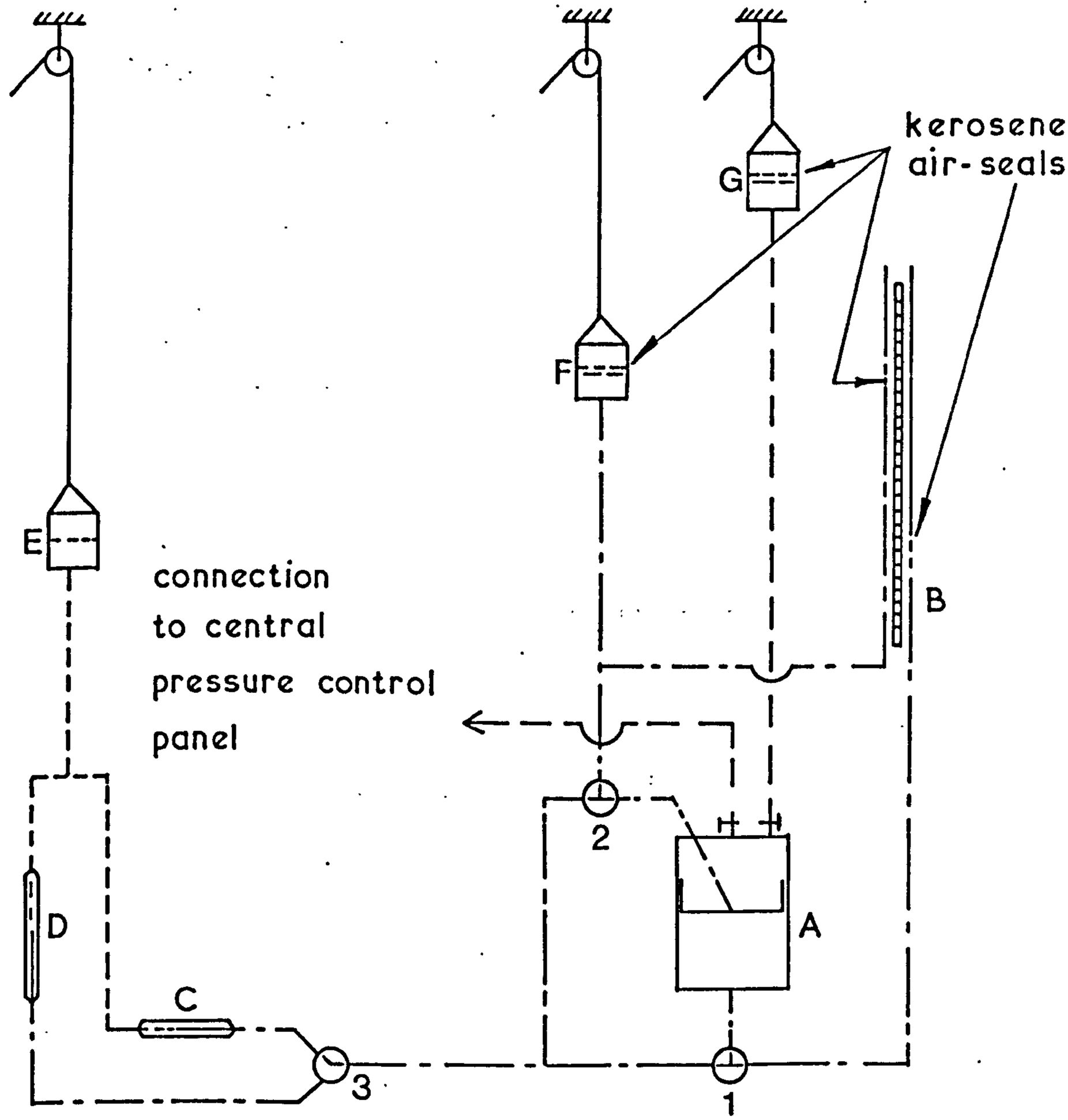
The gauge is filled with dyed kerosene and the flow quantity can be measured from the movement of the kerosene/water interface. The movement of this interface is horizontal and therefore incurs no pressure change and the maximum flow quantity of 0·5cm^3 will produce a change of level in the kerosene reservoir of less than 0·3mm and this is clearly negligible. There is however a pressure drop across the meniscus which forms at the interface due to the difference in the surface tensions of kerosene and water, and pressures must therefore be measured the cell side of the gauge. Table V gives the times for 1cm^3 to flow through samples of different permeability under a unit hydraulic gradient in the cell of area 45·6cm^2.

Permeability cm/s	Time in hours
10^{-5}	0·6
10^{-6}	6
10^{-7}	60

Table V

The peremability of the Dead Sea materials was expected to be in the range 10^{-6}-10^{-7}cm/s and 0·1 and 0·5cm^3 pipettes sub-divided in 0·01cm^3 were therefore used to give a test time of 2-6 hours. The gauge is not suitable for high pressures and it was therefore decided to use a back pressure of only 2 psi for the tests and this had the additional advantage that glass taps could be used in the hydraulic system thus avoiding corrosion problems.

Fig.33 shows the hydraulic system for these tests. The permeameter cell (A) receives its loading initially from a head of fresh water (4 psi) in the reservoir G, and later for higher overburden pressures (6 and 10 psi) from the central pressure control panel shown in Fig.34. The kerosene reservoir E provides a 2 psi back pressure and consequently each sample can be tested under effective stresses of 2, 4, 8 psi. Reservoir F provides differential pressure for permeability tests. The pipette D is used to measure volume changes during consolidation and C is the volume gauge for permeability tests. A general view of the permeameters and flow control system is shown in Fig.35. Because of the very sensitive volume measurements that were to be made the apparatus was installed in a temperature controlled laboratory where the temperature did not vary by more than 1^{o}C during a day.



A consolidation / permeability cell

B standpipes

C pipette 0·1 or 0·5 ml capacity

D pipette 25 ml capacity

—·——·— tubing filled with dead sea water

— — — tubing filled with distilled water

------ tubing filled with kerosene

FIG.33 HYDRAULIC DIAGRAM FOR LOW PRESSURE PERMEABILITY TESTS

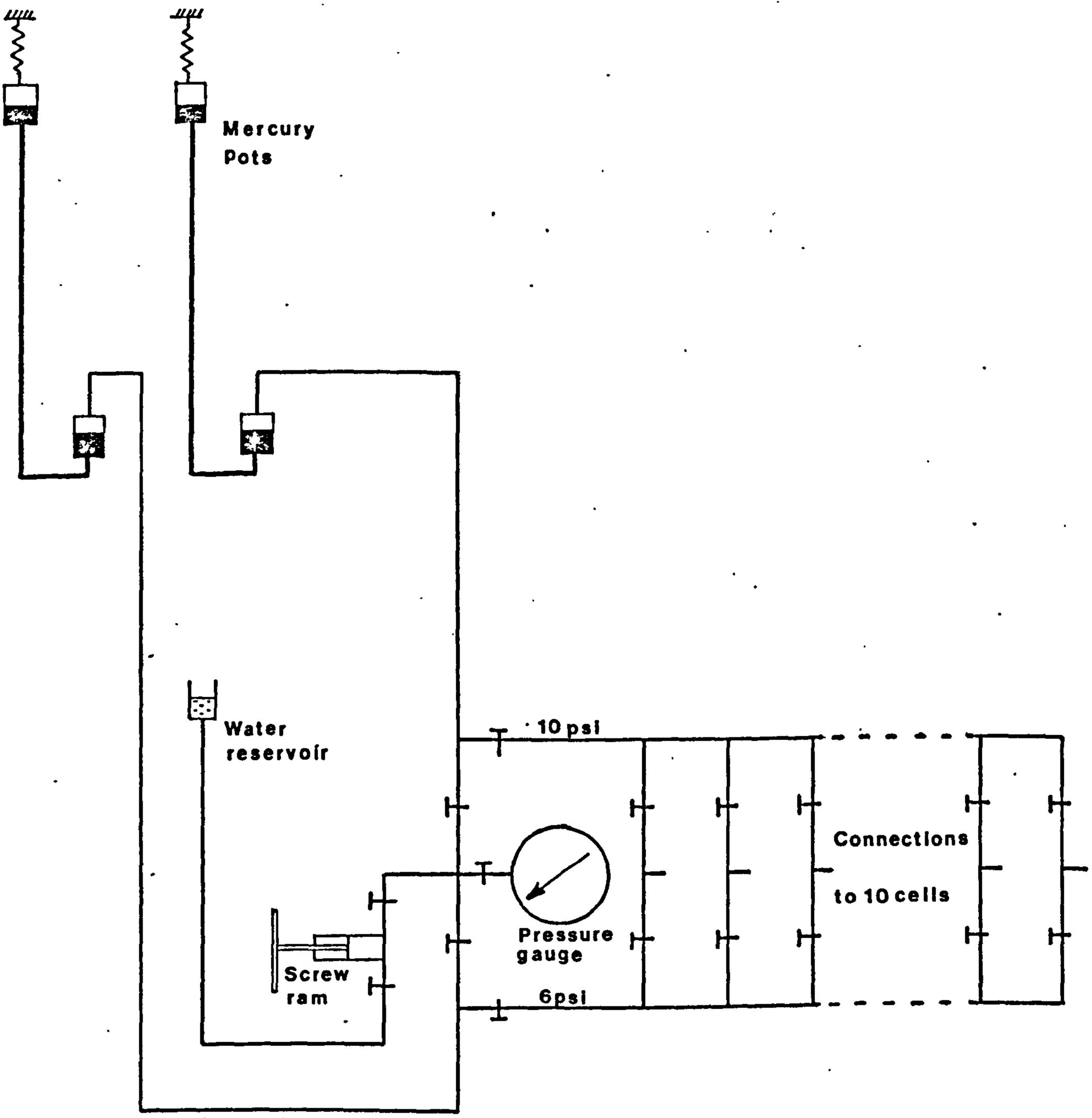


FIG.34 CENTRAL PRESSURE CONTROL PANEL

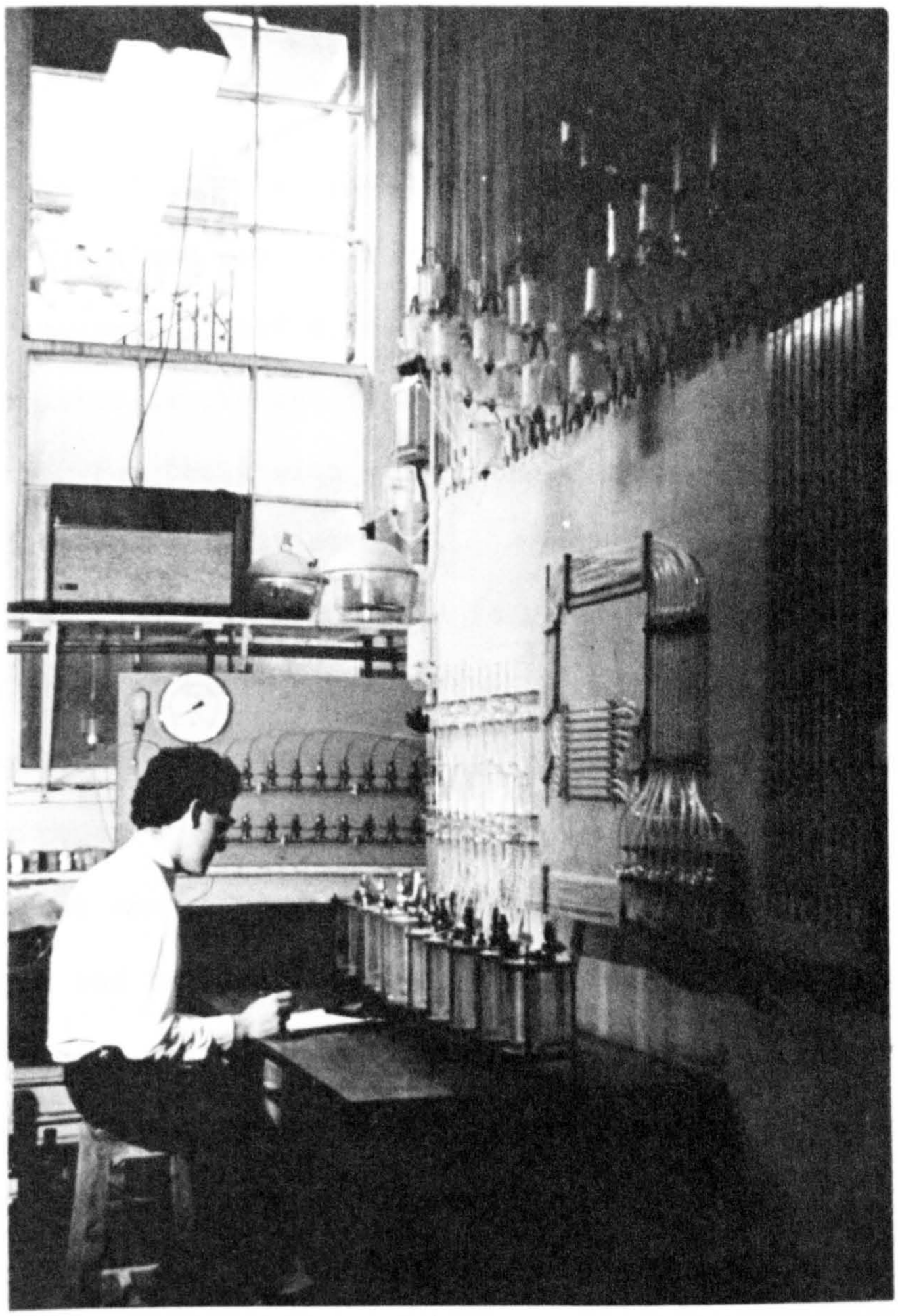


Fig. 35 A GENERAL VIEW OF THE PERMEAMETERS

3.4 Test procedure

a. Sample placement

With the top plate and piston removed the cell was partially filled with Dead Sea water using tap 1 (Fig.33). A saturated porous stone was then placed across the bottom of the cell and covered by a filter paper to prevent fines from the specimen being carried into the stone. For tests with slurry the excess water was removed to leave the filter paper just covered. The cell was then part filled with slurry and soil passing a $\frac{1}{4}$ in (6·35 mm) sieve was poured in slowly to avoid trapping air bubbles. When a soil depth of 7-8cm was reached the excess slurry was removed and the soil surface covered by a filter paper. A second porous stone was then placed and covered with about one centimetre of salt water. The line from the piston to tap 2 was temporarily disconnected so that air and excess water could escape as the piston was inserted. When the piston was seated on the top porous stone this line was reconnected and the top plate bolted down. The space above the piston was then filled with deaired water and the connections to the pressure control panel and reservoir G were made. The height of E was then adjusted so that the back pressure measured with the standpipe was 2 psi. With taps 1, 2 and 3 adjusted to allow double drainage of the sample through pipette D the line to G was opened and consolidation allowed to progress. After forty-eight hours when consolidation was complete (checked by cessation of movement in D) a permeability test was run.

b. The permeability test

With tap 3 turned to pipette C the cell was isolated using taps 1 and 2 and then by adjusting the level of F relative to E and using tap 2 C was just filled with kerosene with tap 1 open to its standpipe. F was then raised until the pressure difference measured on the standpipes was sufficient to produce a hydraulic gradient of about one across the sample. The taps were then adjusted so that the flow was from F through the cell and C to E. This flow was allowed to continue for about two hours whilst the sample equilibrated. The pressure difference shown by the standpipes was then recorded and tap 1 turned to isolate the downstream standpipe (otherwise any flow in or out of the standpipe could upset the readings on C) and volume against time readings taken on C for a further two hours. At the completion of the test the downstream standpipe was again connected and the final pressure difference recorded. This final pressure difference provided a check on the running of the apparatus as no change in pressure difference should have occurred during the test.

The laboratory temperature was monitored throughout the test and if it changed by more than $0{\cdot}5^{o}C$ the test was abandoned as the flow readings would be too greatly disturbed.

c. The consolidation test

When the permeability test was complete the cell was isolated and the pipette D filled with kerosene. The taps were then arranged for double drainage of the sample through D and twenty-four hours allowed for equilibration. At the end of this time the overburden pressure was increased to 6 psi by switching in the appropriate mercury pot of

the central pressure control panel (Fig.34). Volume against time measurements were then made using D. The displacement of the piston could be obtained from the volume measurements and hence the coefficients of consolidation and compressibility of the sample could be calculated. A permeability test was run at this overburden pressure and the sample was then further consolidated under an overburden pressure of 10 psi, and this pressure was maintained throughout the ageing period.

d. Ageing

For each soil/pore fluid combination samples were aged for 1 week, 1 month, 3 months and 1 year. Permeability tests were run at intervals and at the end of the required ageing period the cells were stripped down and consistency and water content measurements made.

e. Calibration of the hydraulic system

After a cell had been stripped down and the sample removed it was reassembled with just the porous stones and filter papers. The flow rate through the cell was then measured for a series of pressure differences between the reservoirs E and F (Fig.33) so that a pressure-flow relationship for the system could be calculated and a correction applied to the heads measured in the permeability tests if necessary. For a typical cell the flow rate was $4{\cdot}6 \times 10^{-3} cm^3/s$ for a pressure difference of 1cm of water. For a sample tested under a gradient of 1 in such a cell the following corrections would apply.

Sample Permeability cm/s	Head correction cm of water
10^{-5}	0·1
10^{-6}	0·01
10^{-7}	0·001

Table VI

It was thus apparent that for the Dead Sea materials which had permeabilities in the range 10^{-6}- 10^{-7}cm/s no head correction was necessary.

f. Consistency measurements

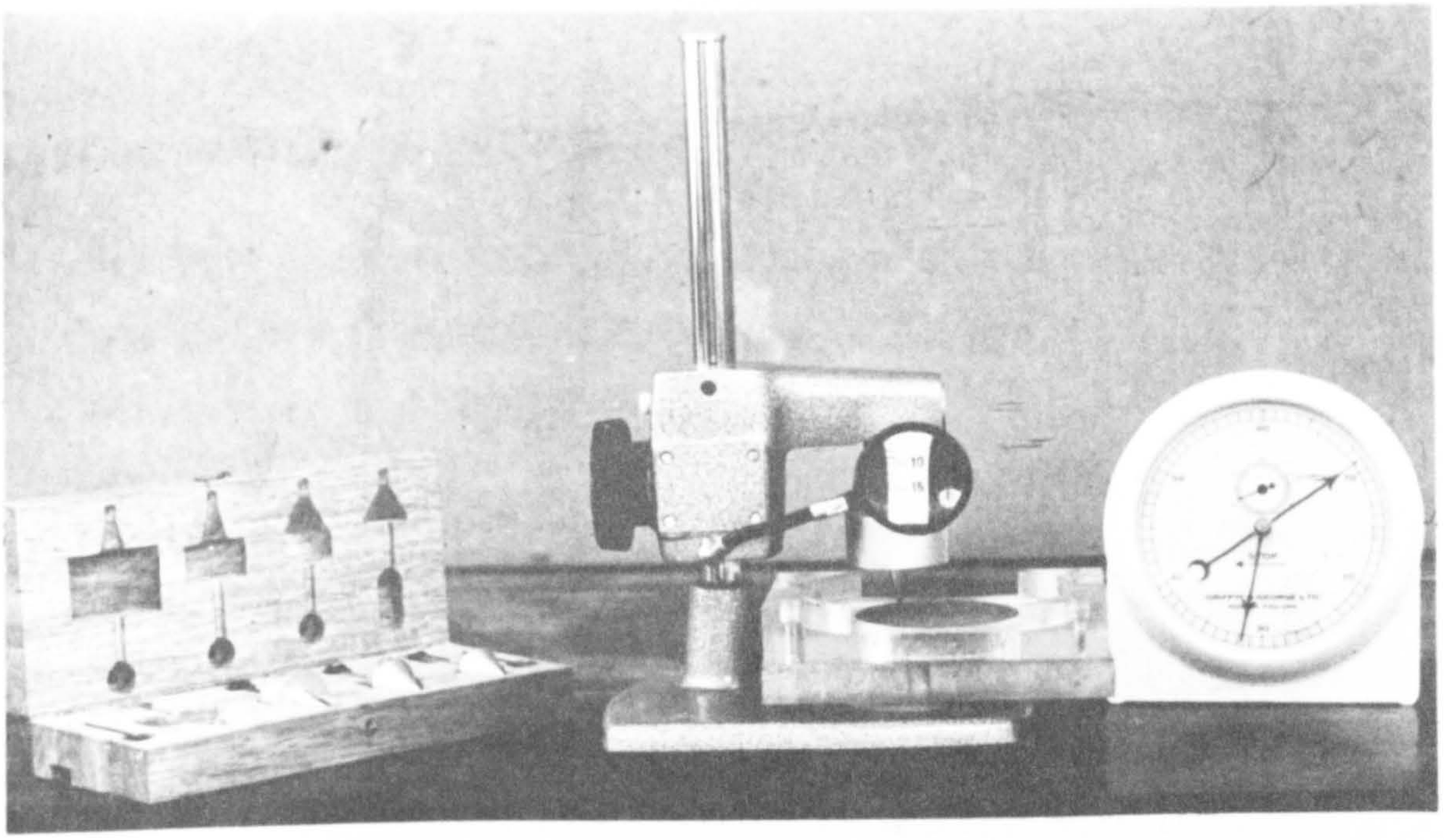
The Swedish Fall-Cone apparatus shown in Fig.36 was used for the consistency measurements as a vane test would have disturbed the samples too much for many tests to be made.

To perform a test the cone supported in the moveable frame is lowered until the point is just touching the surface of the sample. The catch is then operated to move the magnet aside and release the cone. The penetration after thirty seconds is read on the scale (in millimetres) and recorded as the Fall-Cone penetration. Four different cones are supplied with the instrument and the following calibration is provided by the manufacturers*:-

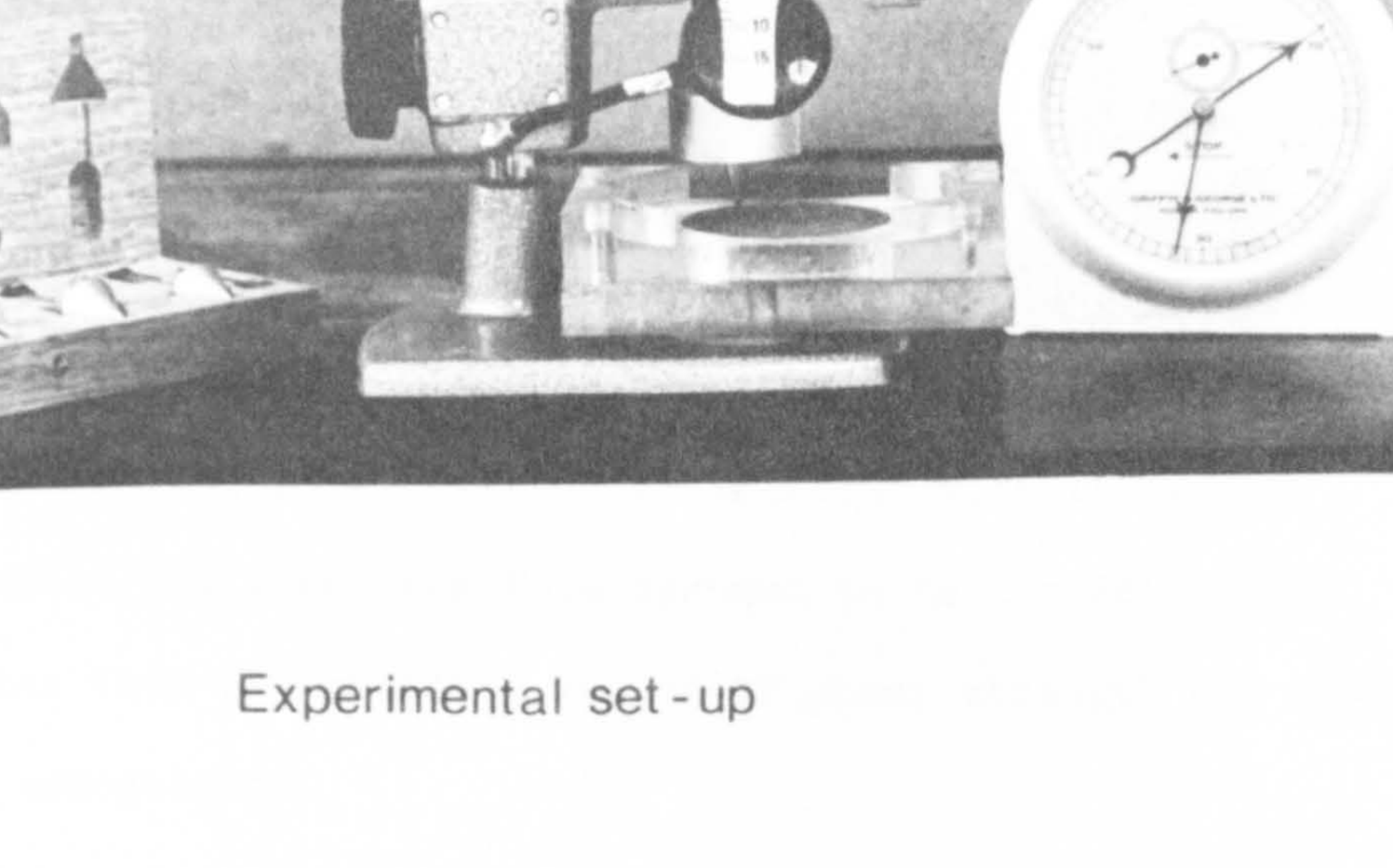
Cone weight grams	Cone apex angle	Penetration mm	Undrained shear strengtht t/m^2
400	30	4-15	25 -1.8
100	30	5-15	4 -0.45
60	60	5-15	0.6-0.067
10	60	5-20	0.1-0.0063

Table VII

* Geonor A/S, Grini Mølle, P.O. Box 99, Røa - Oslo 7, Norway
† $1t/m^2$ = $9·81kN/m^2$



Experimental set-up



Detail of cone and release mechanism

Fig. 36 THE SWEDISH FALL-CONE

The shear strength c_u is inversely proportional to the square of the penetration, h, and thus for the 400 gram cone the relationship between shear strength and penetration is:-

$$c_u(t/m^2) = 400/(h,mm)^2$$

The 400g and 100 g cones were used in the tests and the above calibrations gave reasonable agreement with laboratory shear vane tests, and were therefore assumed to be correct as the relative rather than the absolute values of shear strength are important in assessing homogeneity.

To prepare the permeameter samples for testing a special collar was fitted over the top of the cell and the specimen extruded into it to one third of its height by inserting a rod through the base of the cell. The collar was then cut free using a wire and nineteen Fall-Cone tests performed on the sample in it using a thin Perspex template to mark the positions for the tests. Using the collar three levels from each permeameter sample were tested. Fig.37 shows a typical sample of Dry Clay in Dry Clay slurry before and after testing.

g. Water content measurements

The soil around each Fall-Cone position was removed from the collar with a small sampling tube (16mm diameter) and the water content measured. The salt pore water of the samples complicated this measurement as there was a salt residue left after drying and the weight of water associated with this residue varied considerably with

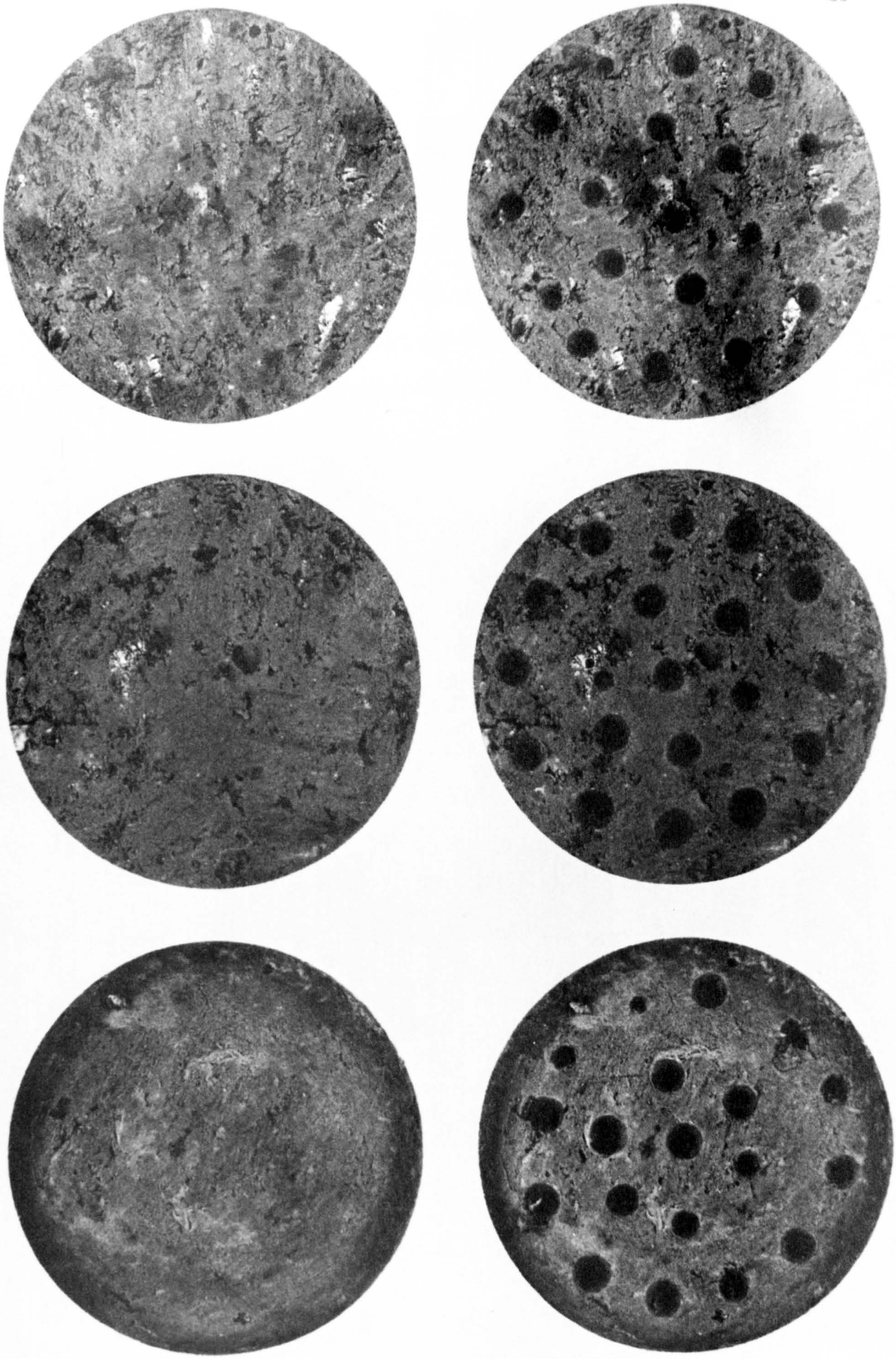


Fig. 37 DRY CLAY IN DRY CLAY SLURRY SPECIMEN

THE THREE LEVELS BEFORE AND AFTER TESTING

drying time and temperature. Further consideration of this problem is made in Appendix 2 but for these tests a standard oven temperature of 105°C (as recommended in the standard specification for soil testing, British Standards Institution,1967) and drying time of twenty-four hours was used. The water contents quoted in this work are apparent water contents ignoring the presence of the salt.

3.5 Test Results

a. Permeability tests

Table VIII shows typical calculated and measured permeabilities for the five samples under the different effective stresses.

Sample Type	Effective stress psi	Measured permeability cm/s x 10^7	Calculated permeability cm/s x 10^7
Dry Clay in Dead Sea water	2	6.8	6.3
	4	4.3	2.7
	8	2.9	
Dry Clay in Dry Clay slurry	2	5.1	4.8
	4	2.5	2.3
	8	2.0	
Dry Clay in Plastic Clay slurry	2	5.0	4.4
	4	3.3	3.9
	8	1.8	
Asphaltic Clay in Dead Sea water	2	17.5	6.7
	4	11.6	9.4
	8	6.0	
Asphaltic Clay in Asphaltic Clay slurry	2	3.1	1.2
	4	2.4	2.0
	8	1.4	

Table VIII

The measured and calculated permeabilities were in reasonable agreement and thus provided a further check on the operation of the apparatus. The permeability dropped considerably as the effective stress was increased and this could be a significant point on site where arching in the trench may limit consolidation.

Fig.38 shows the variation of permeability with time for the samples under an effective stress of 8 psi. For all the Dry Clay samples and the Asphaltic Clay in slurry sample the permeability dropped by a factor of about two during the first ninety days and thereafter there was very little change. The effect of the slurries was to lower the permeability of the samples relative to the pure clays and this effect was most marked for the Asphaltic Clay which did not readily breakdown in water. The wide variation in the measured permeability for this material was probably due to air trapped in the void spaces changing volume as the atmospheric pressure changed. To test this hypothesis, samples of Asphaltic Clay in Dead Sea water and Asphaltic Clay in Asphaltic Clay slurry were tested under a back pressure of 30 psi using a modified hydraulic network. Under this pressure the volume of trapped air should be reduced both by compression and by solution as the solubility of air increases with pressure, and the permeability should therefore be higher. Table IX shows the results of these tests.

Sample Type	Effective stress psi	Measured permeability cm/s x 10^7	Calculated permeability cm/s x 10^7
Asphaltic Clay in Dead Sea water	4	14.1 (11.6)	6.5
	8	12.2 (6.0)	5.4
	16	10.9	
Asphaltic Clay in Asphaltic Clay slurry	4	7.9 (2.4)	8.4
	8	5.9 (1.4)	6.9
	16	2.9	

Table IX

CHANGES OF PERMEABILITY WITH TIME FOR DEAD SEA SOILS

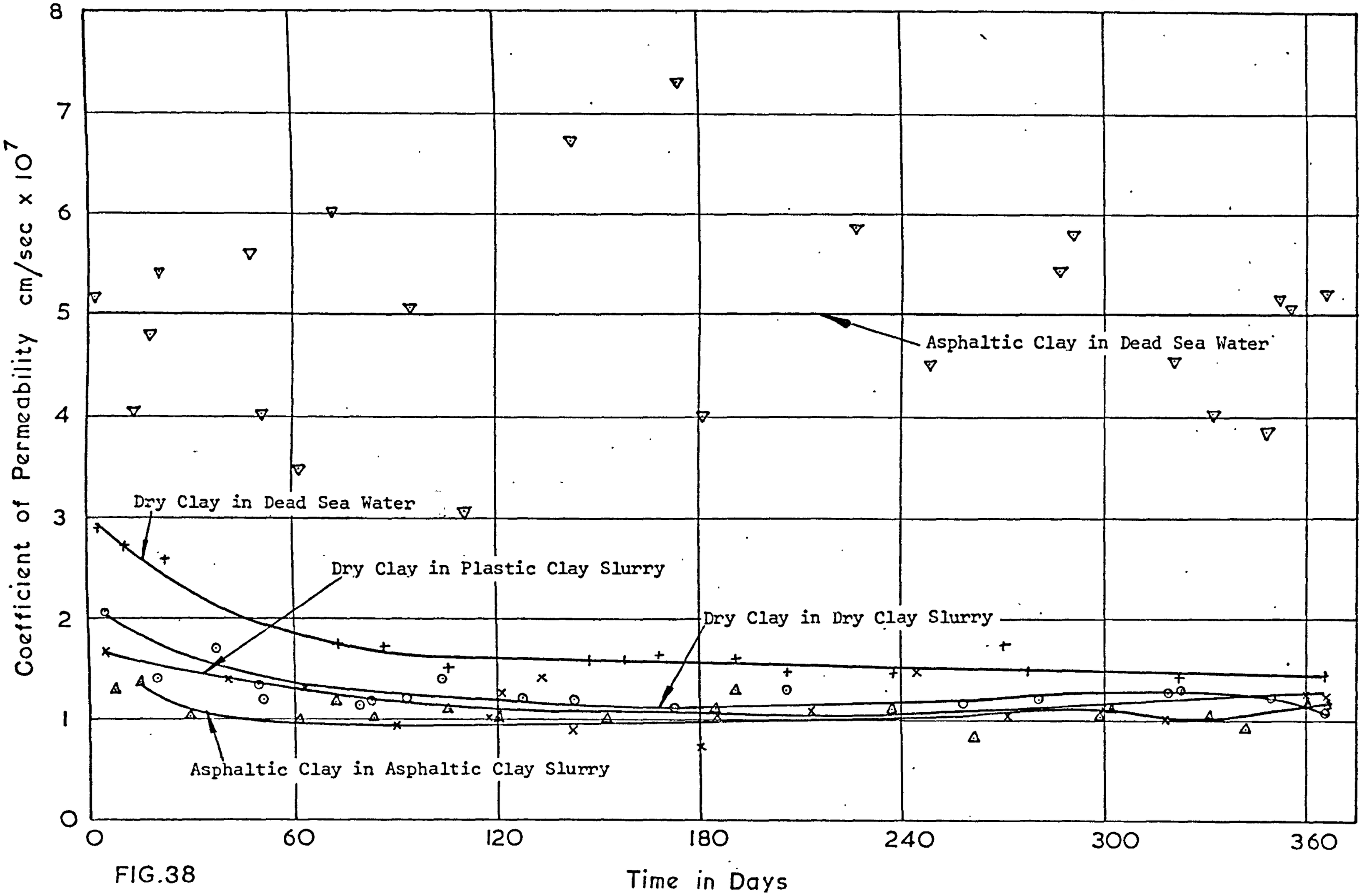


FIG.38

The figures in brackets are those with a 2psi back pressure and it can be seen that permeabilities were markedly higher under the high back pressure especially at the 8 psi vertical stress increment at which there was a factor of about two between the low and high back pressure permeabilities.

b. Consistency and water content tests

Table X summarises the results of these tests. It can be seen that the undrained shear strengths of all the samples increased by a factor of about two over the year and there was a corresponding drop of about 2% in the water contents of the Dry Clay samples, but the Asphaltic Clay showed little change probably because the decrease in water content due to consolidation was offset by the slow penetration of water into the harder clay lumps.

The coefficient of variation for the shear strengths is given as a measure of their variability. It is defined as follows:-

$$\text{coefficient of variation} = \frac{\left[\sum_{i=1}^{n} (x_i - \bar{x})^2/n\right]^{\frac{1}{2}}}{\bar{x}} \times 100$$

$$= \frac{\text{Standard Deviation}}{\text{Mean}} \times 100$$

For the shear strengths this coefficient does not show any trend with ageing and it would therefore appear that the samples do not become more homogeneous with time, but rather that the weak points and strong points gain strength in similar proportions.

Sample Type	Age Months	Water Content %	Bulk density (g/cm^3)	Undrained shear strength (t/m^2)	Coefficient of variation %
Dry Clay in Dead Sea water	¼	23.64	1.85	3.2	33
	1	21.77	2.01	4.5	31
	3	22.33	1.98	3.2	50
	12	21.44	2.06	6.7	42
Dry Clay in Plastic Clay slurry	¼	24.35	1.94	2.2	70
	1	23.85	1.82	2.7	55
	4	23.38	1.97	3.5	37
	12	22.98	2.00	4.6	48
Dry Clay in Dry Clay slurry	¼	26.37	1.88	1.8	62
	1	26.13	1.99	2.5	60
	3	24.80	2.01	2.8	23
	12	24.53	1.98	4.4	54
Asphaltic Clay in Dead Sea water	¼	27.07	1.59	3.4	69
	1	28.66	1.68	2.8	23
	3	26.37	1.72	7.1	42
	12	27.58	1.73	7.9	47
Asphaltic Clay in Asphaltic Clay slurry	¼	31.00	1.55	2.0	28
	1	28.61	1.64	2.9	27
	3	29.06	1.62	4.0	20
	12	30.52	1.83	4.3	25

Change in consistency of Dead Sea soils with time

Table X

CHAPTER FOUR

LARGE SCALE TESTS

4.1 Introduction

4.2 The apparatus

(a) The tank

(b) Flow measuring equipment

4.3 Test procedure

(a) Filling the tank

(b) The crossflow test

(c) The piezometer test

(d) Consistency measurements

4.4 Results of the permeability tests

4.5 Contamination tests

(a) Test procedure

(b) Results of crossflow tests

LARGE SCALE TESTS

4.1 Introduction

Under operating conditions for the Dead Sea Dykes the water level will vary from one side of the Dyke to the other and the efficiency of the cut-off therefore depends on the permeability of the core under crossflow. As it was not feasible to make full scale performance measurements of this nature at the Dead Sea a piezometer test had to be used to measure the permeability of the core. A series of large scale model tests was therefore undertaken to compare the permeability obtained from crossflow and piezometer tests. The tests were carried out in a specially built tank which could accommodate a core of approximately half full size so as to represent trench conditions more closely than the very small scale permeameter tests.

The three clay/slurry combinations used in the previous tests namely Dry Clay in Dry Clay slurry, Dry Clay in Plastic Clay slurry and Asphaltic Clay in Asphaltic Clay slurry were all tested in the tank.

It was also planned to examine the effect on the permeability of the core of contamination by gravel from the walls of the trench.

4.2 The Apparatus

a. The Tank

Fig.39 shows the design and dimensions of the tank. The ends of the tank are lined with porous tiles* to act as filters and these can be connected either to a reservoir of Dead Sea water or for crossflow tests to flow measuring equipment. One side of the tank is made from 2cm thick Perspex as an observation window and the other side is fitted with 18 small bore water level tubes (standpipes) so that pore pressures in the soil can be monitored. Fig.40 shows a photograph of the tank and reservoir.

Because of the large size of the tank these tests were carried out in the laboratory of a commercial soils testing contractor. The work was supervised by Professor Nash but only the Dry Clay tests were performed by the Author.

b. Flow measuring equipment

(i) Crossflow test

To perform a crossflow test the water level at one end of the tank is lowered relative to the other end. During this draw-down the upper part of the sample has to drain and some consolidation occurs so that the outflow from the sample is initially greater than the inflow to the sample. It is therefore necessary to monitor both inflow and outflow as crossflow equilibrium cannot be said to exist until these are approximately equal.

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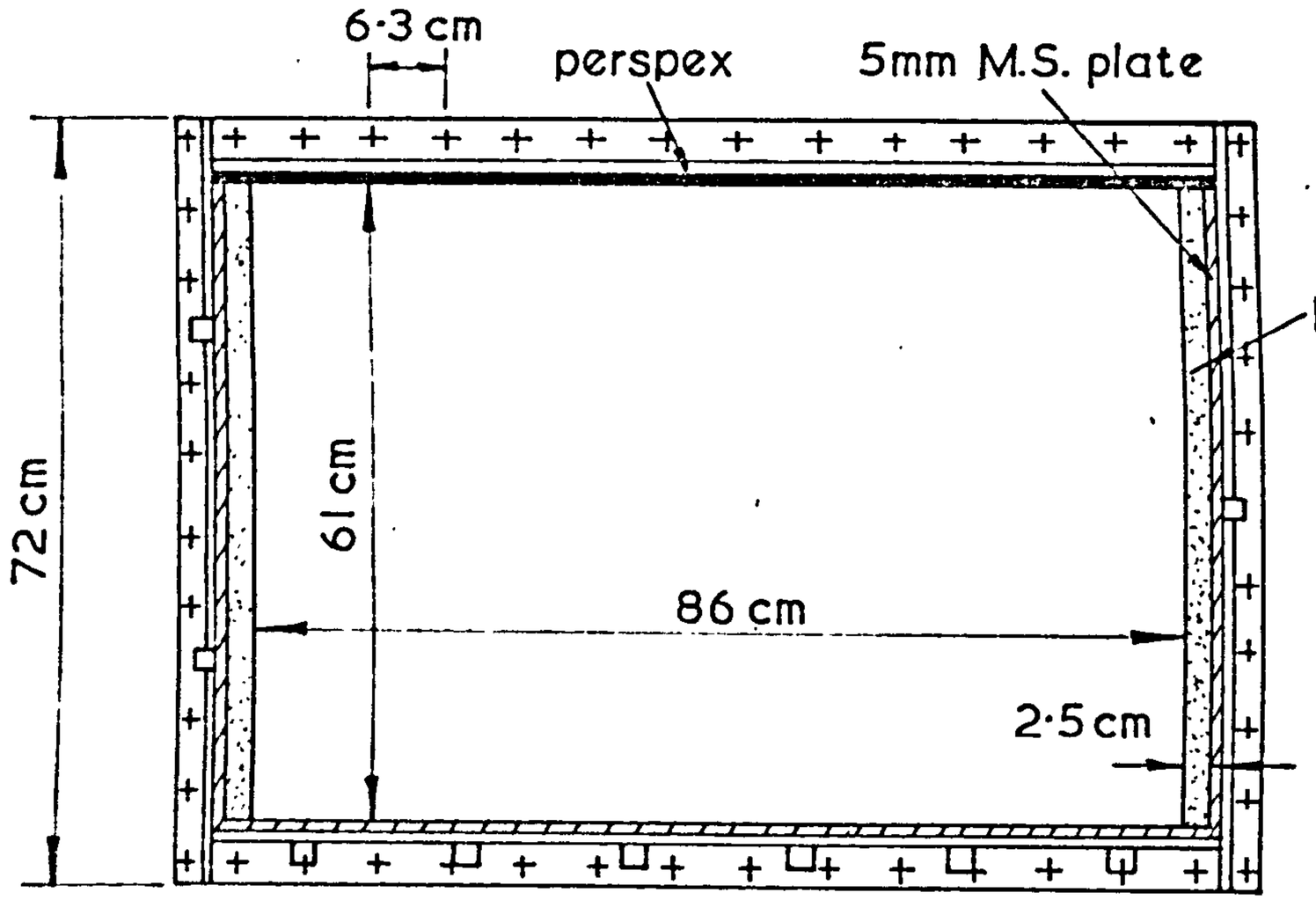
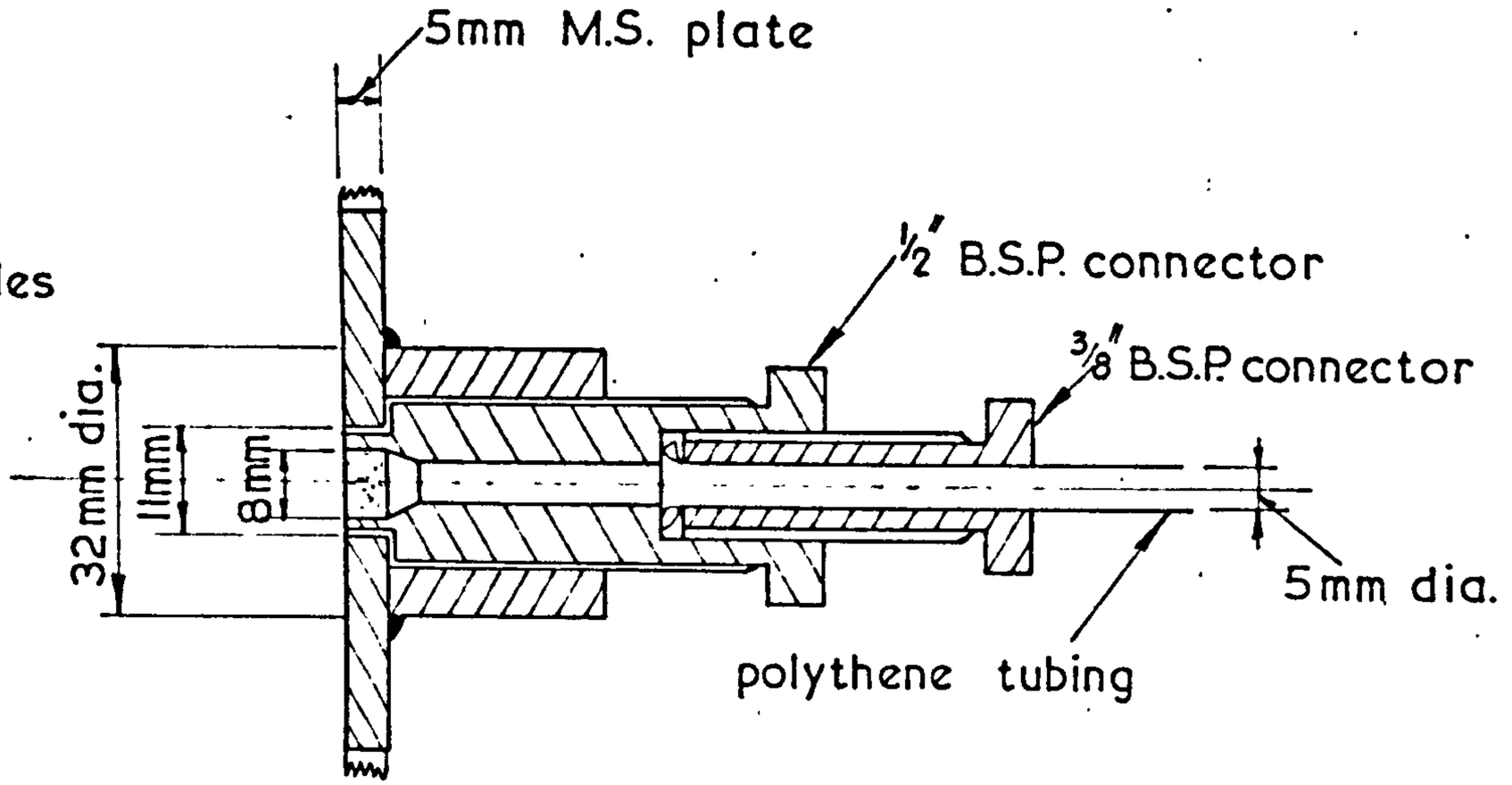
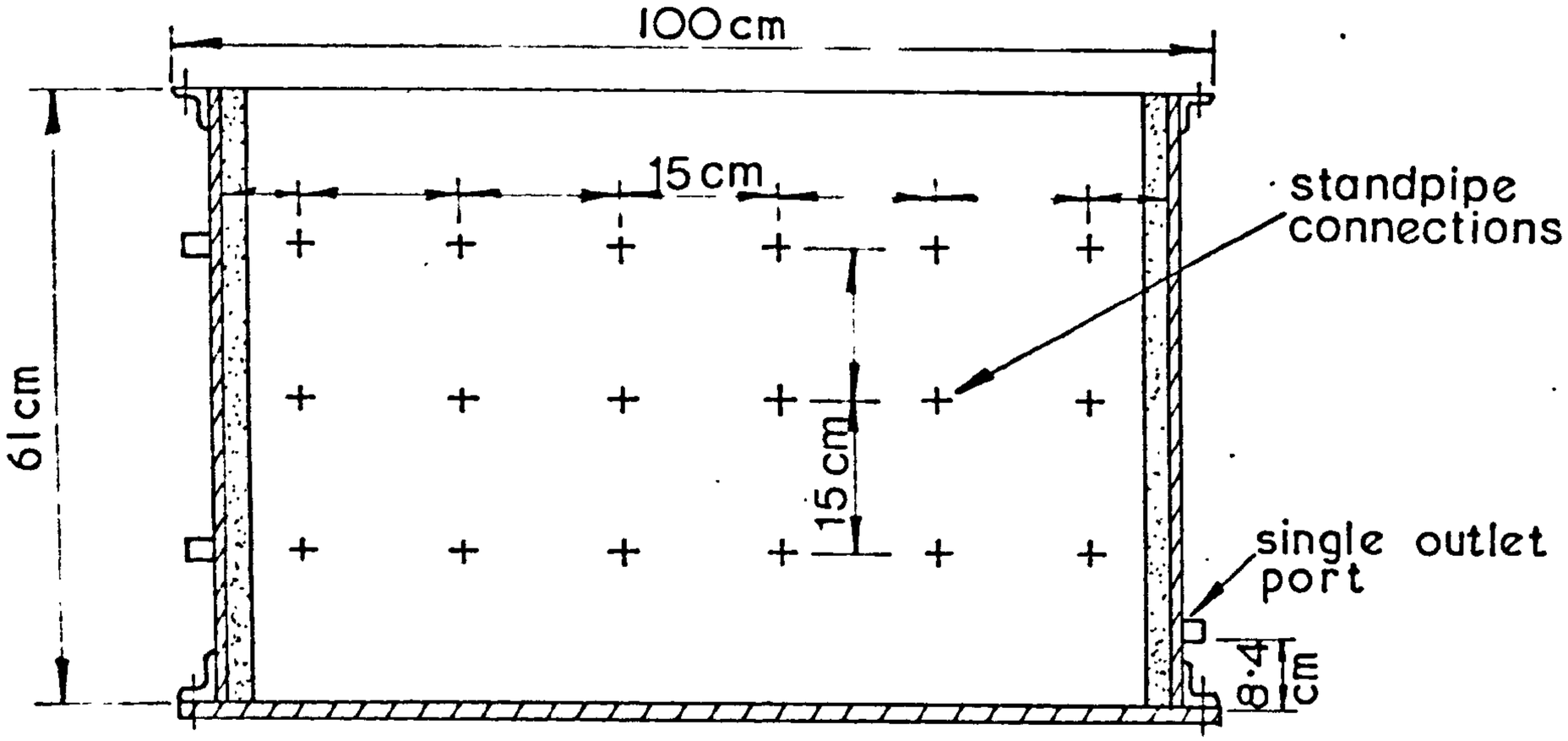
* obtained from Doulton Industrial Products, Stone, Staffordshire, Pyrolith grade G26-28



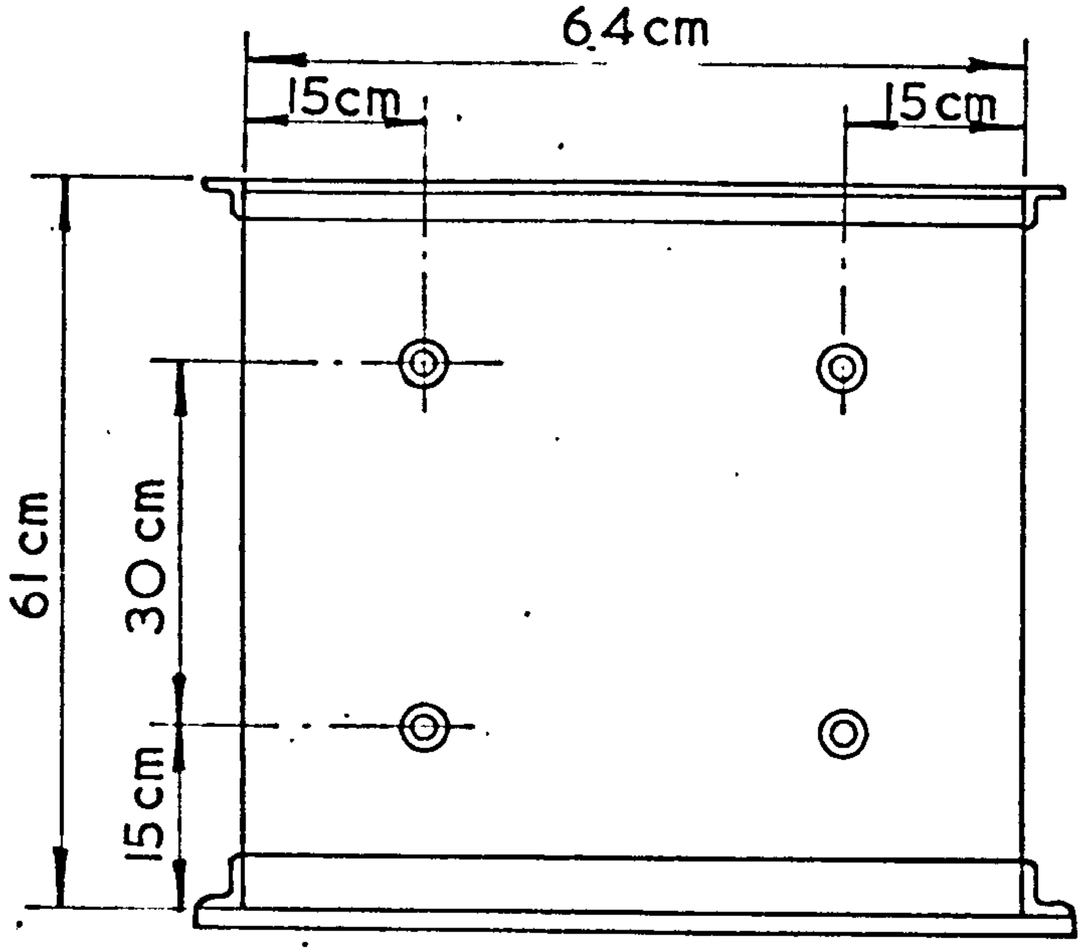
SECTIONAL PLAN



DETAIL OF STANDPIPE CONNECTION



SECTIONAL ELEVATION



INLET END ELEVATION

Fig. 39 STEEL TANK FOR CROSSFLOW AND PIEZOMETER TESTS

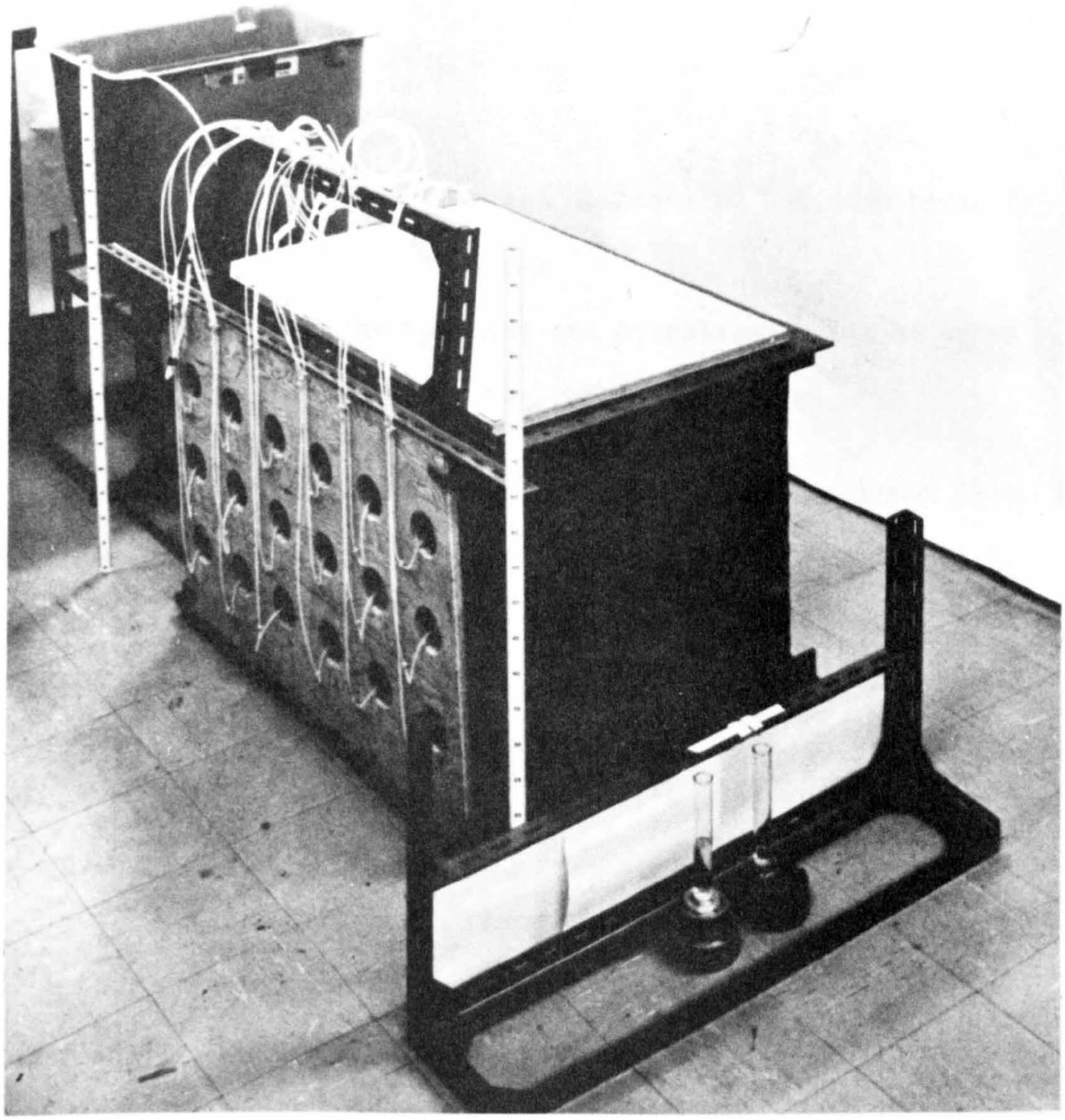


Fig. 40 GENERAL ARRANGEMENT FOR LARGE SCALE TESTS

The flow rate through the sample can be calculated from Dupuit's equation:-

$$q = \frac{kx}{2L}(h_1^2 - h_2^2) \qquad 4.1$$

(Harr, 1962)

where L is the distance from the upstream to the downstream face

x is the width of the tank

h_1 and h_2 are the upstream and downstream heads of water respectively, above the impermeable base.

The permeability of the material in the tank was not known in advance of the tests except that since it had only been consolidated by self weight the permeability would be less than that measured in the permeameters with an 8 psi overburden pressure. The flow gauges therefore had to be designed to operate over rather a wide range of flow rates.

Table XI shows the theoretical crossflow in the tank with the typical upstream and downstream levels of 55 cm and 15 cm respectively, for various assumed values of the permeability.

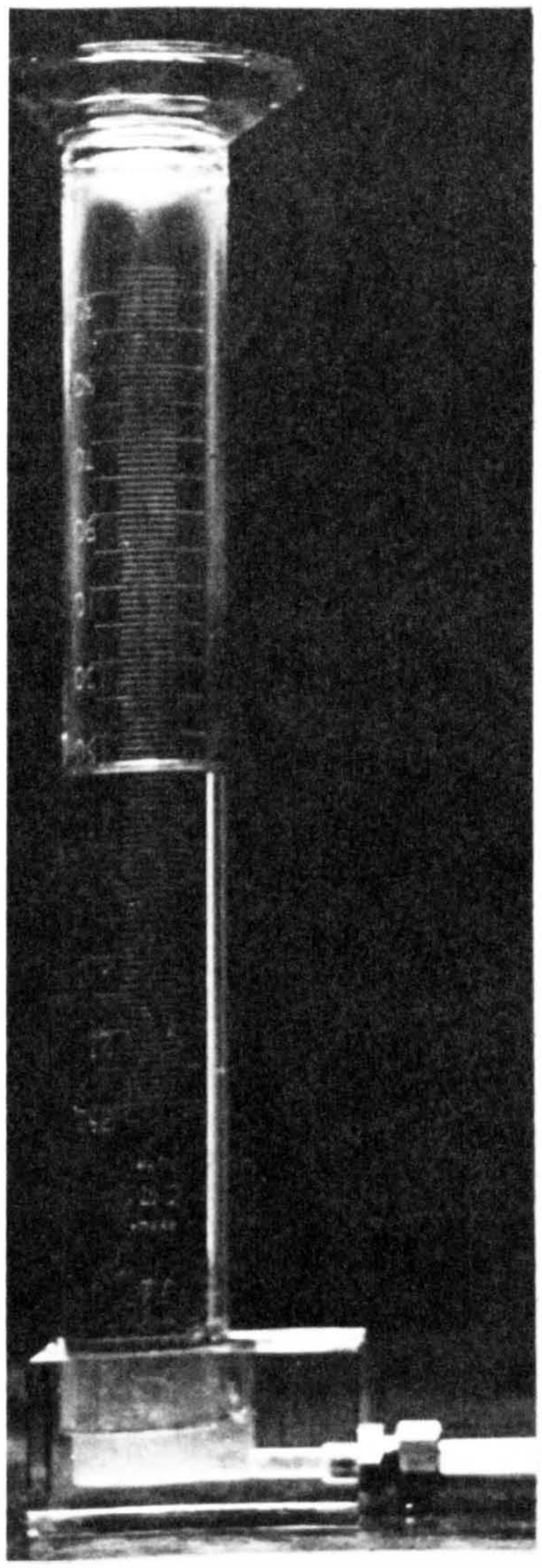
Permeability cm/s	Total flow cm^3/hr
10^{-5}	36
10^{-6}	3.6
10^{-7}	.36

Table XI

Quantity of crossflow for various assumed values of permeability

For outflow tests these quantities can be directly measured with a measuring cylinder or graduated pipette if the flow is collected in a beaker. The outflow gauge thus consisted of a slotted angle frame to hold the flexible discharge pipe at a fixed level over a beaker containing a small amount of kerosene which floated on the water surface and thus prevented evaporation. At intervals dependent on the flow rate the beaker was removed and the volume of flow measured.

The inflow gauge presented a greater problem as any change in water level produced by the gauge would cause considerable fluctuations in flow as the level in the porous stones changed. The commercial volume-change gauge (Fig.31) in which the flow was measured by the vertical movement of a kerosene/water interface was thus unsuitable and the horizontal capillary flow gauge (Fig.32) would have only allowed the flow to be monitored for rather short intervals at these flow rates. After some experimentation the gauge shown in Fig.41 was developed by the contractors and used for the inflow measurements. The gauge consists of a trough connected to the upstream porous stones into which an inverted measuring cylinder can be fitted. Initially the cylinder is filled with water but if the level in the trough drops below the lip of the cylinder air can enter and water escapes until the lip is once again covered. Slight fluctuations in the water level can occur as the flow from the trough is continuous whereas the flow from the cylinder occurs as discrete pulses as the bubbles enter, but the bubbles are typically of about $0.25cm^3$ and the water thus displaced will cause an insignificant change in the water level in the trough and porous stones.



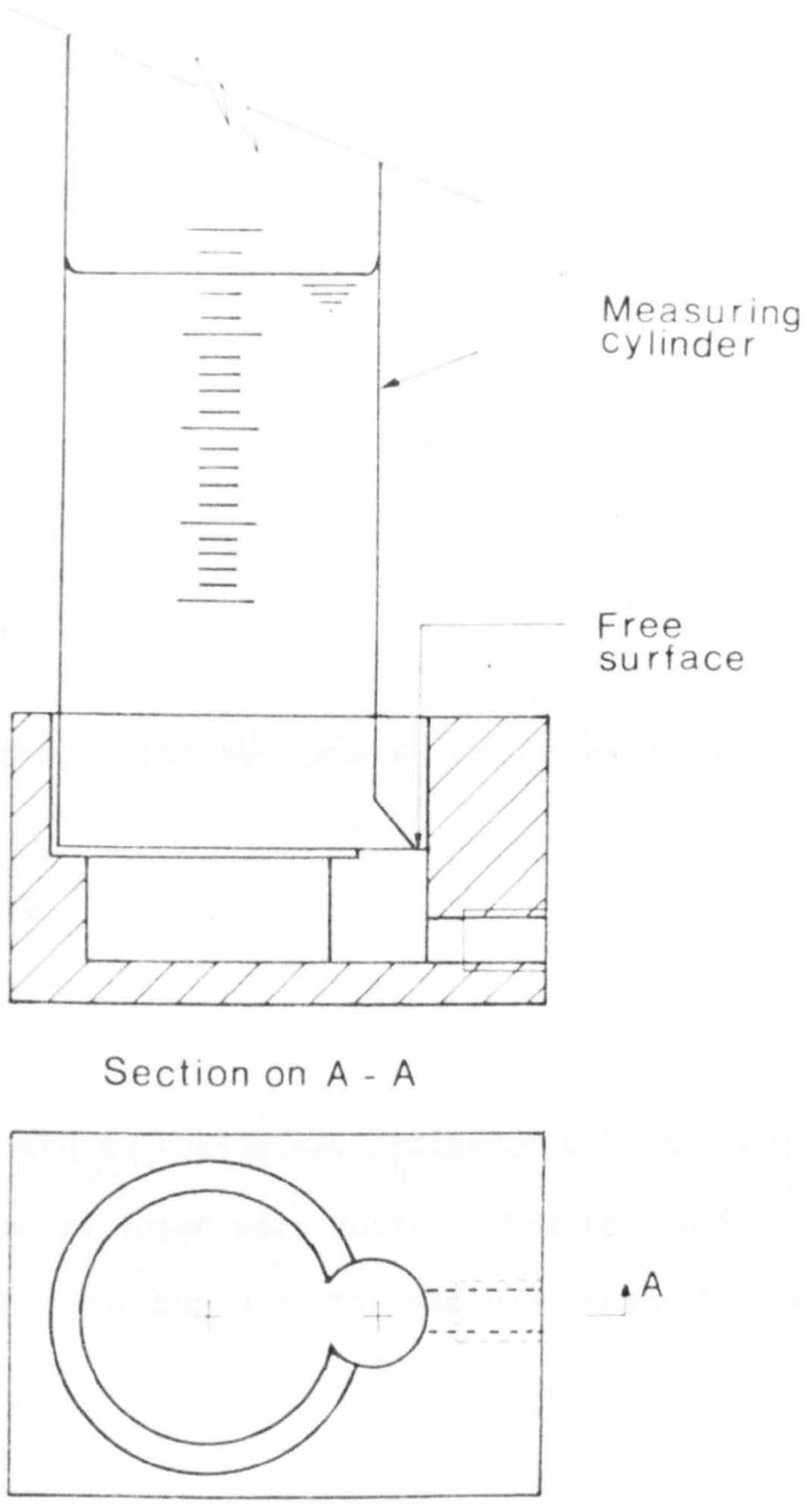


Fig. 41 CONSTANT HEAD FLOW MEASURING APPARATUS

A 250cm^3 measuring cylinder was used for most of the tests and the gauge could therefore operate continuously for many hours.

(ii) Piezometer tests

Al Dhahir (1967) gives a full analysis of the determination of permeability using piezometers. For a piezometer with a length to diameter ratio of about 1 the flow rate can be calculated from the formula by Gibson (1963)

$$q = 4\pi rk \frac{\Delta u}{\gamma_w} \qquad 4.2$$

where r is the radius of a sphere with the same surface area as the piezometer

Δu is the excess pressure

γ_w is the density of water

The piezometers used had an equivalent radius of 0.43 cm and excess pressures of up to 10 cm of water were used. The theoretical flow from the piezometer for this maximum head for various permeabilities is shown in table XII.

Permeability cm/s	flow rate cm^3/hr
10^{-5}	1.9
10^{-6}	0.19
10^{-7}	0.019

Table XII

The piezometer tests were to be short term tests and for such tests these flow rates could conveniently be measured with a horizontal capillary gauge of 0·5cm^3 capacity. The gauge as shown in Fig.32 was of very simple design and this was modified for more general use. Fig.42 shows the modified gauge. The capillary tube is mounted between two Perspex blocks to which the flow connections are made. The reservoir in block A is filled with coloured kerosene and to start a test the gauge is tipped momentarily so that a small quantity of kerosene flows up the inclined section, then with valve C closed valve D is opened slightly to allow the kerosene to flow into the capillary up to the zero graduation. At the end of a test the kerosene collects in the reservoir in block B and can be drawn off as necessary.

The pressure drop in the gauge, piezometer, and associated pipework was measured prior to the tests by allowing the piezometer to discharge freely under water and measuring the flow rate at a series of applied heads. The relationship between pressure drop and flow rate was found to be as follows:-

$$\Delta H = 0{\cdot}26q + 1{\cdot}3 \qquad 4.3$$

where ΔH is the head loss in cm of water

q is the flow rate in cm^3/hr.

The first term is the correction for viscous flow in the pipework and is negligible for permeabilities below 10^{-6}cm/s. The 1·3 cm of water required to produce flow results from the pressure drop across the menisci of the kerosene bubble in the capillary, and is a function of the difference in surface tensions of the water and kerosene and the diameter

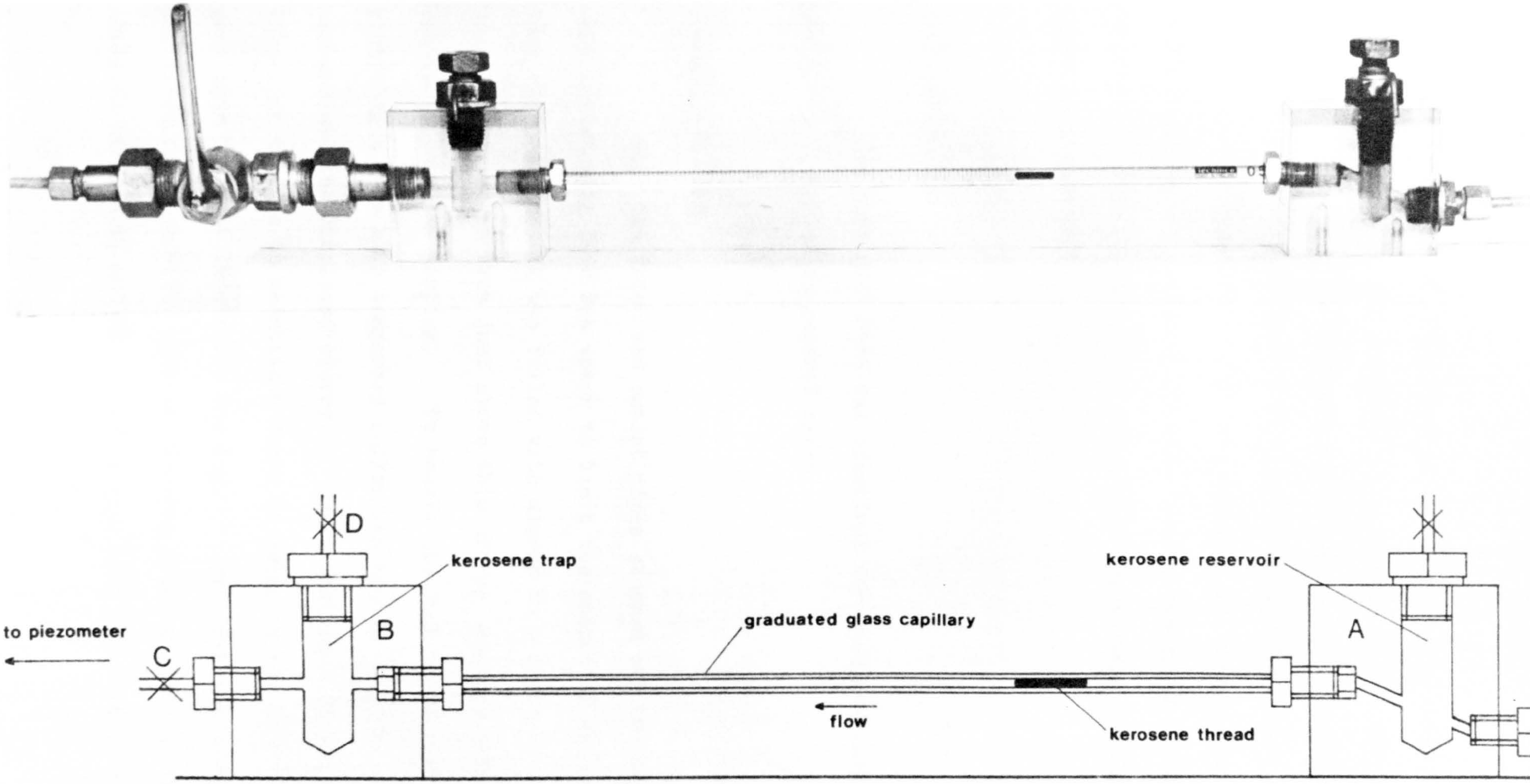


Fig.42 THE HORIZONTAL FLOW GAUGE

of the tube. In the tests described in Chapter 3 these corrections were not required as the pressure was measured between the sample and the gauge, but this was somewhat cumbersome as the standpipe had first to be allowed to equalise and then be isolated before flow measurements could start. Whereas if the head corrections are applied pressures differences can be measured directly between reservoir levels without recourse to standpipes, with a consequent improvement in accuracy and speed of measurement.

4.3 Test procedure

The test procedure was identical for each material and is therefore described in general terms.

a. Filling the Tank

With the inlet and outlet pipes stopped off the porous stones were wetted with Dead Sea water to limit the amount of material drawn into them as the tank was filled with slurry to a depth of 30cm. Clay was then poured in from just above this slurry surface and allowed to settle without any tamping. To enable the tank to be completely filled with clay a 40cm high temporary collar was bolted on to the top to accommodate the displaced slurry. Small additions of slurry were made from time to time to maintain a depth of 30cm, as the tank was filled with clay to about 10cm above the top. The clay was placed as delivered from the site, that is with the grading shown by the dry sieve analyses of Figs.27 and 28. After placement the sample was allowed to

settle for twenty-four hours, the free slurry was then siphoned off and the excess clay removed so that the surface was flush with the top of the tank which was then covered by a waterproof Saran sheet. The material added to and removed from the sample was weighed so that the bulk density could be calculated.

The level in the reservoir was then adjusted to be about 5cm below the soil surface and the inlet and outlet pipes of the tank then linked to it. At this stage the 18 standpipes all showed excess pore-water pressures about 15cm above the reservoir level. This pressure resulted in drainage to the porous stones and this was essentially complete after a seven day ageing period.

b. <u>The Crossflow test</u>

At the end of the ageing period when the excess pore pressures had dissipated a crossflow test was run. The downstream connection to the reservoir was undone and the outflow pipe clamped in the slotted angle frame 15cm above the base of the tank, so that water could drain freely from it. No flow readings were taken for the first four hours as the sample and porous stones drained but thereafter readings were taken at regular intervals.

The phreatic surface during crossflow can be deduced from Dupuit's equation (4.1) for a homogeneous medium. For any section a distance y from the upstream filter where the pressure is h_1 the pore pressure h is given by:-

$$h = \sqrt{h_1^2 - (h_1^2 - h_2^2)y/L} \qquad 4.4$$

After about 3-4 days the standpipes showed approximately this theoretical distribution and the inflow gauge was connected so that the inflow could be monitored. In general crossflow equilibrium was not achieved but readings were continued until the inflow and outflow rates were sufficiently close for an average value of the permeability to be calculated. Fig.43a shows a plot of the flow rates for Dry Clay in Plastic Clay slurry and it can be seen how slow was the movement towards equilibrium.

The downstream level was then raised to 35cm and a second check crossflow run. This raising of the water level produces swelling and as this occurs more rapidly than consolidation it was hoped that crossflow equilibrium would also be achieved more rapidly but as can be seen from Fig.43b the effect was marginal.

c. <u>The Piezometer tests</u>

At the end of the second crossflow test the tank was once again connected to the reservoir and the water level restored to about 5cm below the soil surface. The sample was allowed to settle for a week and then the $\frac{1}{4}$ inch model piezometer shown in Fig.44 was gently pushed into the soil at the centre of the tank to a depth of 30cm, and left for one day with the water pressure in it equal to that in the reservoir.

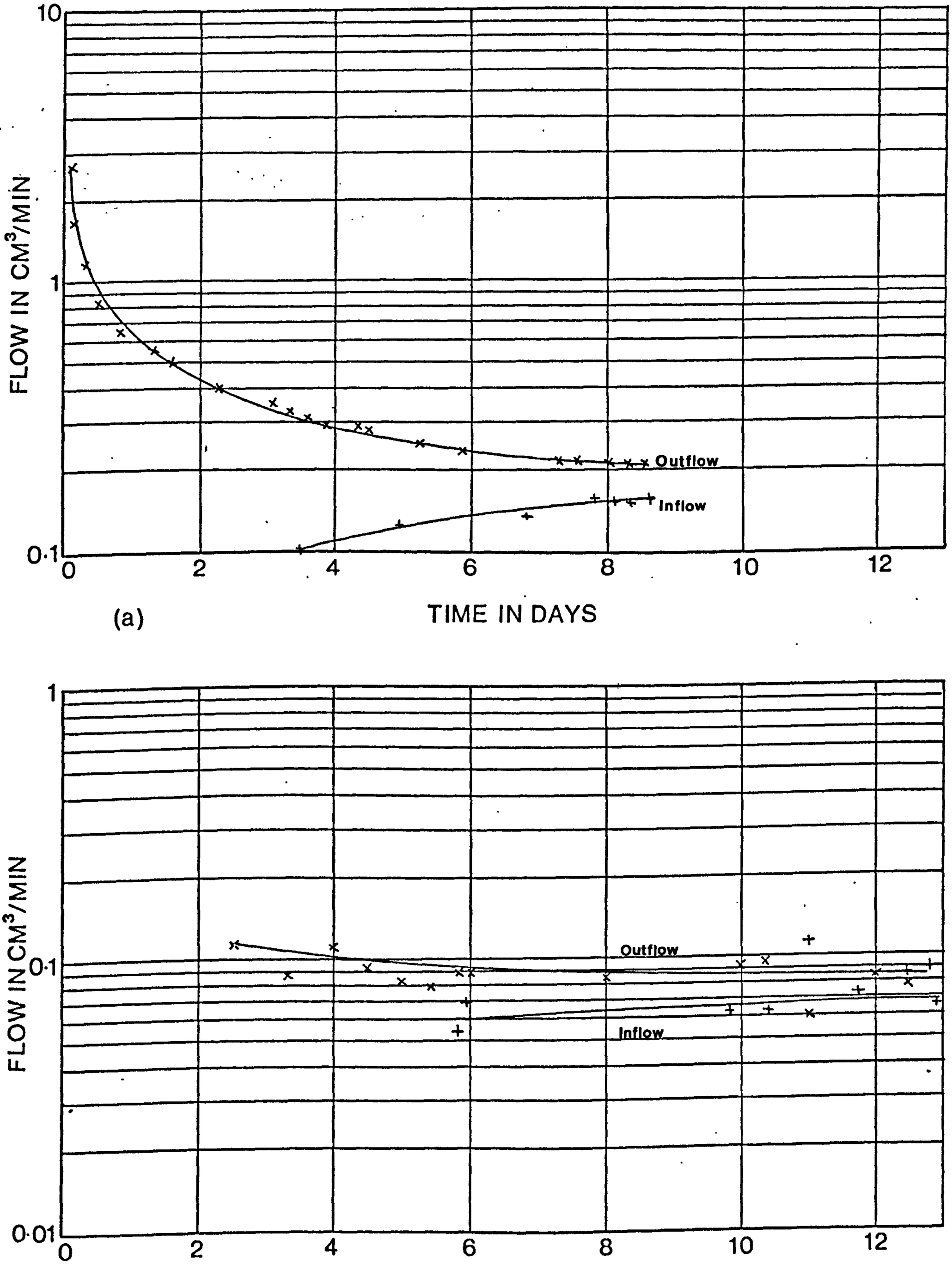


Fig. 43

CROSSFLOW RATES FOR DRY CLAY IN PLASTIC CLAY SLURRY

(a) First crossflow

(b) Second crossflow

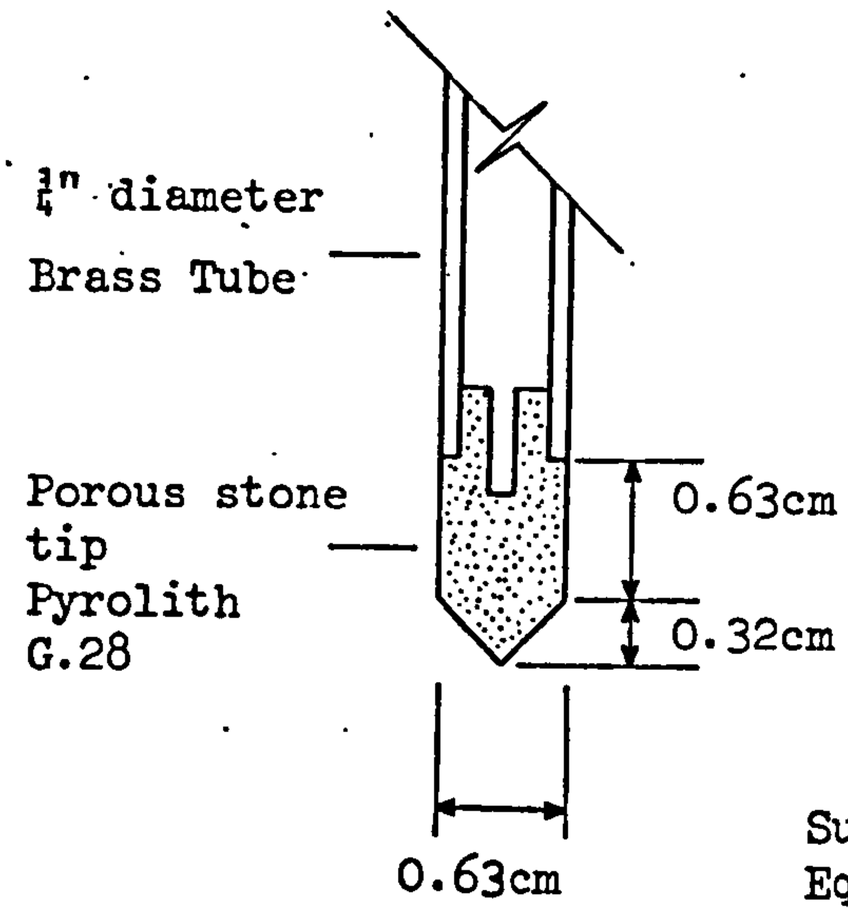


Figure 44 The ¼" Piezometer

The pressure was then raised by 5cm and the flow rate measured over a period of two hours. The test was then repeated with an excess head of 10cm and for the Dry Clay tests a further four sets of tests were done at the quarter points of the tank. The Asphaltic Clay sample was tested only at the centre but excess pressures up to 60cm were used, and some hydraulic fracturing probably occurred as the critical excess pressure for compressible soils as given by Bjerrum, Nash, Kennard and Gibson (1972) is approximately:-

$$\frac{\Delta u}{p_o'} = 0.4 \qquad 4.5$$

where p_o' is the effective overburden pressure

Δu is the excess pressure

Thus for a piezometer at a depth of 30cm in a compressible soil the critical excess pore pressure is about 12cm of water. If this pressure is exceeded the soil may crack with a consequent considerable increase in permeability.

d. Consistency measurements

On the completion of the permeability tests routine classification and identification tests were carried out at depths of 8cm, 30cm and 53cm as the soil was removed from the tank. The weight of soil removed from the tank was also recorded so that the final bulk density could be calculated. The results of these tests are summarised in Table XIII.

4.4 Results of the permeability tests

The results of the crossflow and piezometer tests are summarised in Table XIV. The permeabilities were all considerably higher than those measured in the permeameters due to the low effective stresses in the tank.

Sample type	First crossflow h_1 = 55cm h_2 = 15cm	Second crossflow h_1 = 55cm h_2 = 35cm	Piezometer test - average values at Δu = 5cm	Piezometer test - average values at Δu = 10cm
Dry Clay in Dry Clay slurry	4.9	3.6	2.8	3.6
Dry Clay in Plastic Clay slurry	2.9	2.1	1.6	2.3
Asphaltic Clay in Asphaltic Clay slurry	1.1	0.9	2.3	1.9

Permeability of Dead Sea materials
(cm/s x 10^6)

Table XIV

Sample Type	Level below surface (cm)	Water content (%)	Bulk density (g/cm^3)	Undrained shear strength (t/m^2)			ϕ_d (c_d = o)	Coefficient of consolidation (cm^2/s)x10^5
				Geonor vane	Torvane	Fall-cone		
Dry Clay in Dry Clay Slurry	8	30.4	1.94	0.43	0.37	0.41		
	30	28.9	1.94	0.50	0.39	-		
	53	27.3	1.96	0.85	1.12	1.00		
		28.9	*1.95*	*0.59*	*0.63*	*0.70*	*33°*	*9.4*
Dry Clay in Plastic Clay Slurry	8	23.9	1.97	0.47	0.54	0.82		
	30	27.4	2.00	0.54	0.58	0.81		
	53	25.7	2.00	0.80	0.76	0.93		
		25.7	*1.99*	*0.60*	*0.63*	*0.85*	*33°*	*9.7*
Asphaltic Clay in Asphaltic Clay Slurry	8	36.1	1.80	0.41	0.42	0.68		
	30	39.8	1.78	0.38	0.35	0.57		
	53	36.1	1.85	0.29	0.29	0.65		
		37.3	*1.81*	*0.36*	*0.35*	*0.63*	*38°*	*8.8*

TABLE XIII

Properties of various soils determined while stripping tanks

average values are shown in italics

From the results in Table XIV it can be seen that the average values of permeability obtained from the piezometer tests were in reasonable agreement with those of the crossflow tests, though as shown in Table XV there was a significant variation in the results of the individual piezometer tests.

Sample type	Excess head cm	Position 1	2	3	4	5
Dry Clay in Dry Clay Slurry	5	5.4	2.1	1.1	2.6	2.8
	10	6.5	1.5	1.9	4.3	4.0
Dry Clay in Plastic Clay Slurry	5	0.9	2.3	1.2	2.5	1.3
	10	1.4	2.7	2.2	3.6	1.6

Permeability ($cm/s \times 10^6$) of Dry Clay from Piezometer tests

Table XV

For each position the agreement between the results at the two pressure increments was good, and it appears that zones of different permeability exist within the sample probably due to the lumps in the original material. For the Asphaltic Clay sample the excess pressure was raised in increments to 60cm, reduced to 10cm and then increased again to 40cm.

Excess head cm	First test series	second test series
5	2.3	
10	1.9	11
15	3.3	
20	2.4	17
30	6.0	
40	16.0	27
50	21.7	
60	25.0	

Permeability (cm/s x 10^6) of Asphaltic Clay from piezometer tests

Table XVI

From the results of the Asphaltic Clay tests it can be seen that the permeability increased markedly at the higher excess heads, and the second test series in the same position showed that this increase was permanent. A full analysis of this effect is given by Kennard (1971).

The piezometer test thus appears to give reasonable results for the permeability of the Dead Sea soils provided the excess pressure is limited.

4.5 Contamination tests

These tests were to assess the effect on the permeability of a cut-off of gravel from the walls of the trench falling in and becoming trapped in the core. Two types of contamination were tested, first when the gravel formed isolated unconnected pockets and second when it formed a continuous layer through the core.

a. Test procedure

Following the tests on Dry Clay in Dry Clay slurry the soil was only partly removed to leave a depth of 30cm, the collar was then refitted and the tank filled with slurry to a depth of 40cm. Three isolated pockets of $\frac{3}{4}$in-$\frac{3}{16}$in gravel each weighing 1·3kg were dropped through the slurry onto the surface of the clay. The edge of one of these pockets was about 13cm from the centre of the observation window and the other two were about the same distance from the far corners. The tank was then refilled with the clay that had been removed plus some fresh Dry Clay and allowed to age before carrying out crossflow tests similar to those previously described. After two crossflow tests the tank was once again stripped down to the 30cm level and the same grading of gravel deposited through slurry to form a 10cm thick layer right across the tank. After ageing two crossflow tests were again performed.

b. Results of crossflow tests

The results of the crossflow tests are summarised in Table XVII.

Type of inclusion	First crossflow	Second crossflow
None	4.9	3.6
3 isolated pockets	1.6	1.3
Continuous layer	4.2	3.1

Crossflow permeabilities ($cm/s \times 10^6$) with and without gravel inclusions

Table XVII

The permeability with the gravel pockets was lower than with no contamination and this must have been due to remoulding of the clay during the refilling of the tank, with the gravel having little effect on the permeability. The permeability with the continuous gravel band was very similar to the original value and it appears that the slurry effectively sealed the gravel. This was a somewhat unexpected result and it was therefore decided to examine the sealing properties of slurries in greater detail.

CHAPTER FIVE

THE PERMEABILITY OF A SLURRY

5.1 Piezometer tests

(a) Introduction

(b) The apparatus

(c) Test procedure

(d) Test results

5.2 Constant head permeability tests

(a) The apparatus and test procedure

(b) Test results

5.3 Bentonite slurries

(a) Selection of slurry concentration

(b) Test results

5.4 Prediction of failure gradients

THE PERMEABILITY OF A SLURRY

5.1 Piezometer tests

a. Introduction

The results of the crossflow tests with the continuous gravel band can be explained either if the slurry remains fluid in the gravel voids and flows under the applied hydraulic gradient to form a filter cake on the downstream porous stone or if the slurry gels in the voids and restricts the flow of water in much the same manner as a clay. Permeability reduction by filter cake formation will be characterised by an initially high flow rate which will fall as the filter cake is formed and consolidates in position. That is, the measured permeability will decrease time, whereas if the slurry sets in the voids the permeability will be independent of time. These two mechanisms of permeability reduction could not be distinguished from the results of the crossflow tests as any initial high flow rate would have been masked by the draining of the sample at the start of drawdown and as the tests were continued only until approximate crossflow equilibrium was achieved no definite trend in permeability could be established. Further tests were therefore needed to establish the mechanism of permeability reduction, and a series of piezometer tests was undertaken as this could provide a sensitive indication of the behaviour of the slurry. In a piezometer test if the slurry does remain fluid the effective radius of the piezometer will increase during inflow to the sample as a void forms round the tip. During outflow from the sample the slurry will move towards the piezometer to form a filter cake.

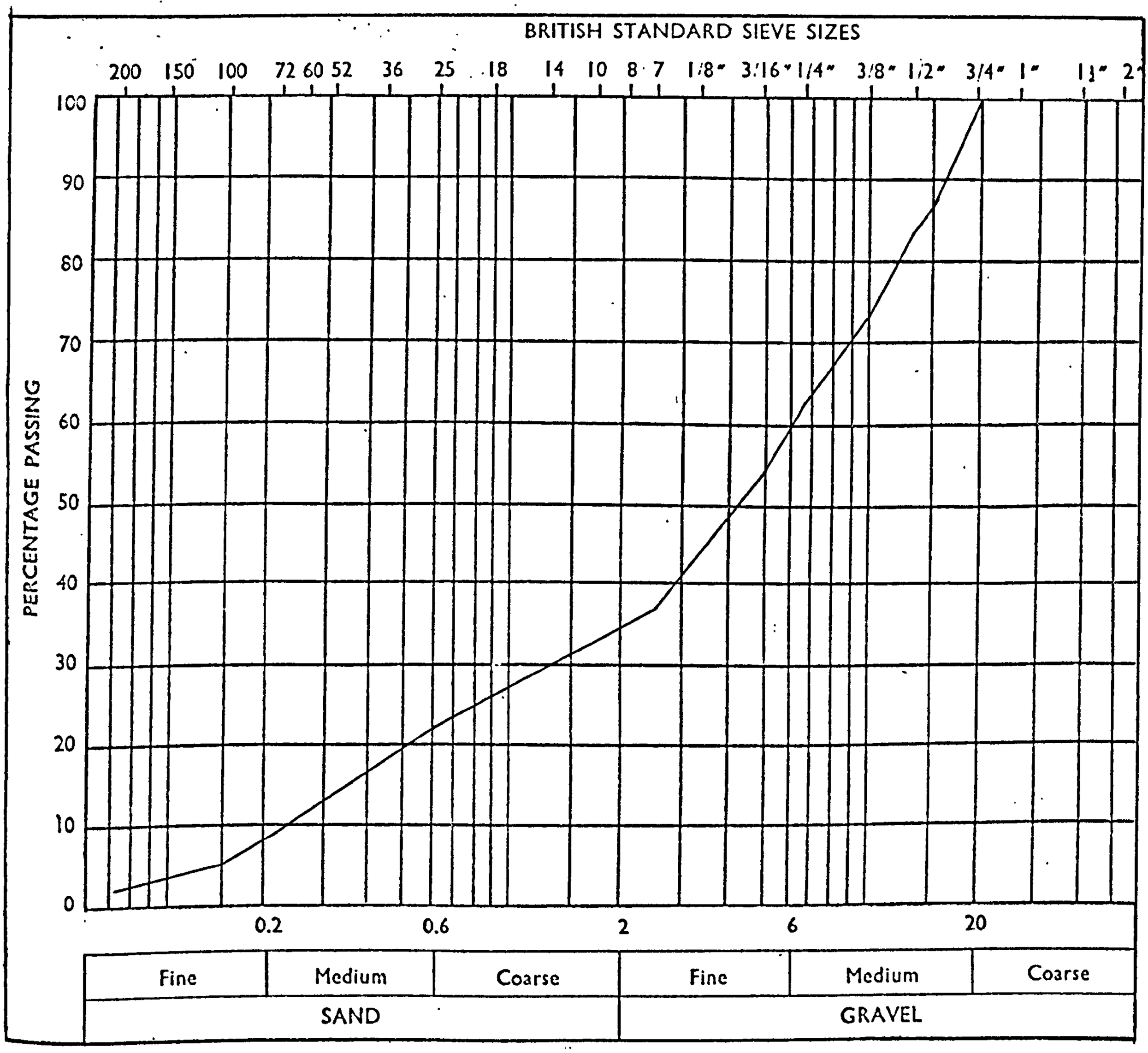
Thus the measured permeability will increase with inflow and decrease with outflow.

b. The apparatus

The tests were carried out in a tank similar in design to that used for the large scale tests. The tank was 40 x 40 x 30cm deep and fitted with porous tiles across two opposite faces, which could be connected to a reservoir of Dead Sea water. The piezometer used was $\frac{3}{4}$in (1.9cm) in diameter and had an equivalent radius of 1.54cm. Flow rates were measured with the horizontal flow gauge (Fig.42) as in the previous piezometer tests. The head-flow characteristic of the gauge, piezometer and associated pipework was measured prior to the tests so that a correction could be made for pressure drop in the system.

c. Test procedure

The procedure for filling and testing was very similar to that used for the large scale tests. The tank was part filled with Dry Clay slurry and the piezometer clamped in position at the centre. Passing $\frac{3}{4}$in gravel/sand with the grading shown in Fig.45 was then added a little at a time and carefully tamped down. When the tank was filled with the slurry/gravel/sand mixture it was covered with a polythene sheet and allowed to age for three days with the water levels in the reservoir and piezometer system at the soil surface. The pressure in the piezometer was then raised by 0.5cm and the corresponding inflow rate measured over a period of three hours. The test was then repeated



GRADING CURVE FOR GRAVEL

USED IN PIEZOMETER TESTS

Fig.45

at an excess pressure of 1cm and thereafter in increments of about 1cm to 5cm. The pressure was then returned to the reservoir level and a further three days allowed for equilibration. . The flow rate was then measured at 0.5cm and 1cm and thereafter at increments of about 1cm to 6cm below the reservoir level. The equilibration procedure was then repeated and the pressure raised to 5cm above the reservoir level and then in increments until the sample failed, presumably by cracking though the failure surface could not be identified because of the weakness of the material.

The tests were then repeated using $\frac{3}{4}$in-No.7 (ASTM) (19.05-2.83mm) fraction of the gravel. The procedure for this test was very similar to that of the previous test except that the outflow rate was measured first as it was feared that the slurry might fail under a very low inflow head in the coarser gravel.

d. Test results

The results of the tests on the two materials are summarised in Figs.46 and 47 in which the flow rates are plotted against the corrected heads (corrected for head loss in the system). It can be seen that in both cases the flow rate is independent of the direction of flow and depends only on the magnitude of the applied head. That is, the inflow and outflow permeabilities of the slurry/gravel mixtures are equal. The measured permeabilities are summarised below with the permeabilities of the untreated gravel for comparison.

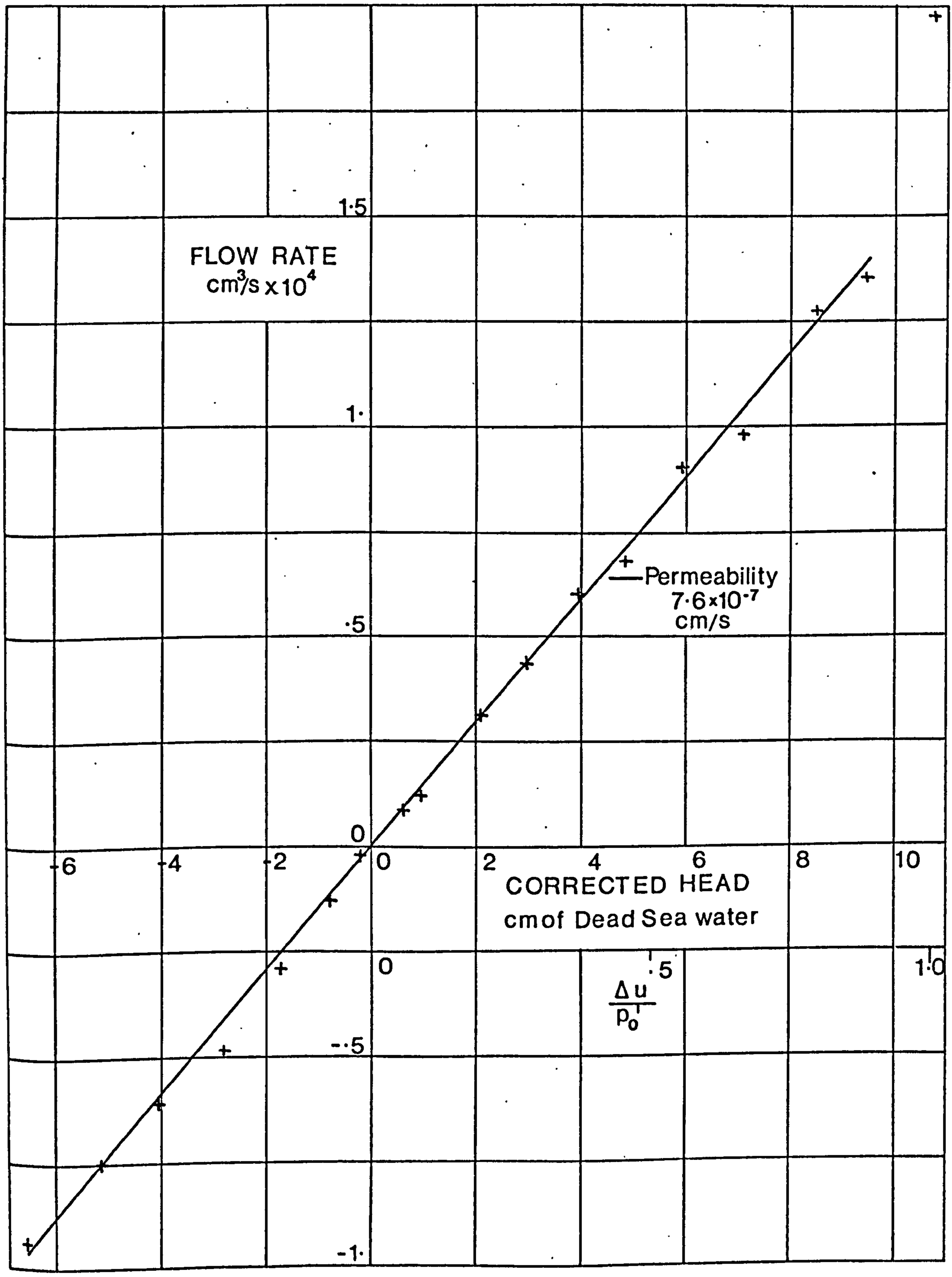


Fig. 46 PIEZOMETER TESTS WITH SLURRY IN PASSING 3/4" GRAVEL

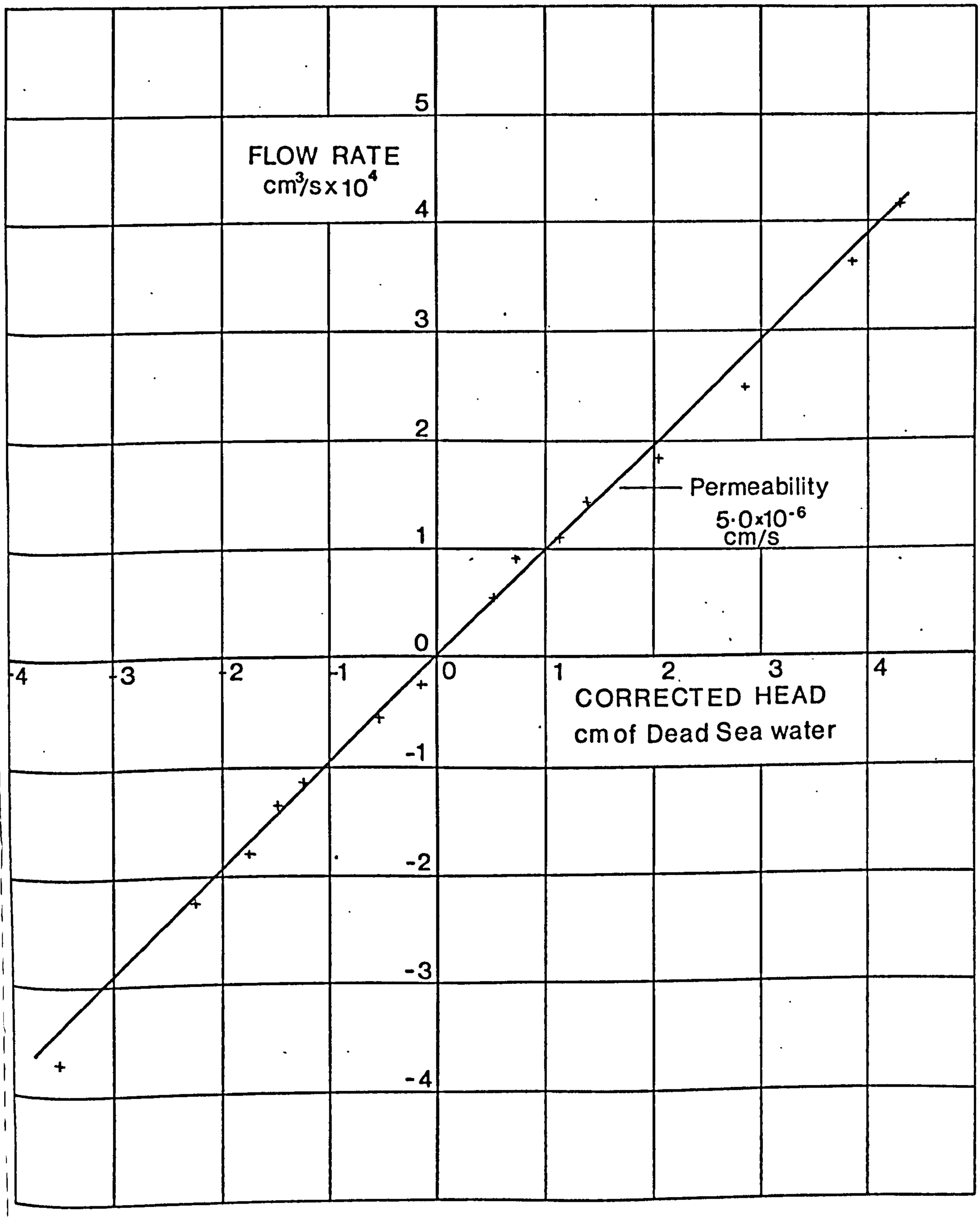


Fig. 47 PIEZOMETER TESTS WITH SLURRY IN 3/4"- No 7 (ASTM) GRAVEL

	slurry/gravel cm/s	gravel without slurry cm/s
passing $\frac{3}{4}$in	7.6×10^{-7}	3×10^{-4}
$\frac{3}{4}$in-No.7 (ASTM)	5.0×10^{-6}	~5

For the passing $\frac{3}{4}$in material a value of p_o' can be calculated assuming the material is homogeneous. A scale of $\frac{\Delta u}{p_o'}$ is included in Fig.46 and it can be seen that the sample failed at

$$\frac{\Delta u}{p_o'} = 1$$

which is the value predicted for an incompressible material by Bjerrum et al (1972). However for the $\frac{3}{4}$in - No.7(ASTM) gravel no estimate of p_o' can be made as the gravel forms a continuous structure with the slurry supported in the voids. This sample was not taken up to failure. In addition to these tests a sample of slurry was tested in a wide beaker with no gravel to see if the slurry alone could withstand any pressure difference. Fig.48 shows a plot of head against flow rate for this set up. The outflow permeability was 4.2×10^{-6} cm/s (similar to that for the slurry in the coarser gravel) and showed no variation with applied head. The inflow permeability was 1.7×10^{-5}cm/s but this must be regarded as a very approximate value as failure occurred at head of 0.13cm and at this level very slight errors in pressure measurement will cause considerable errors in the calculated value of the permeability.

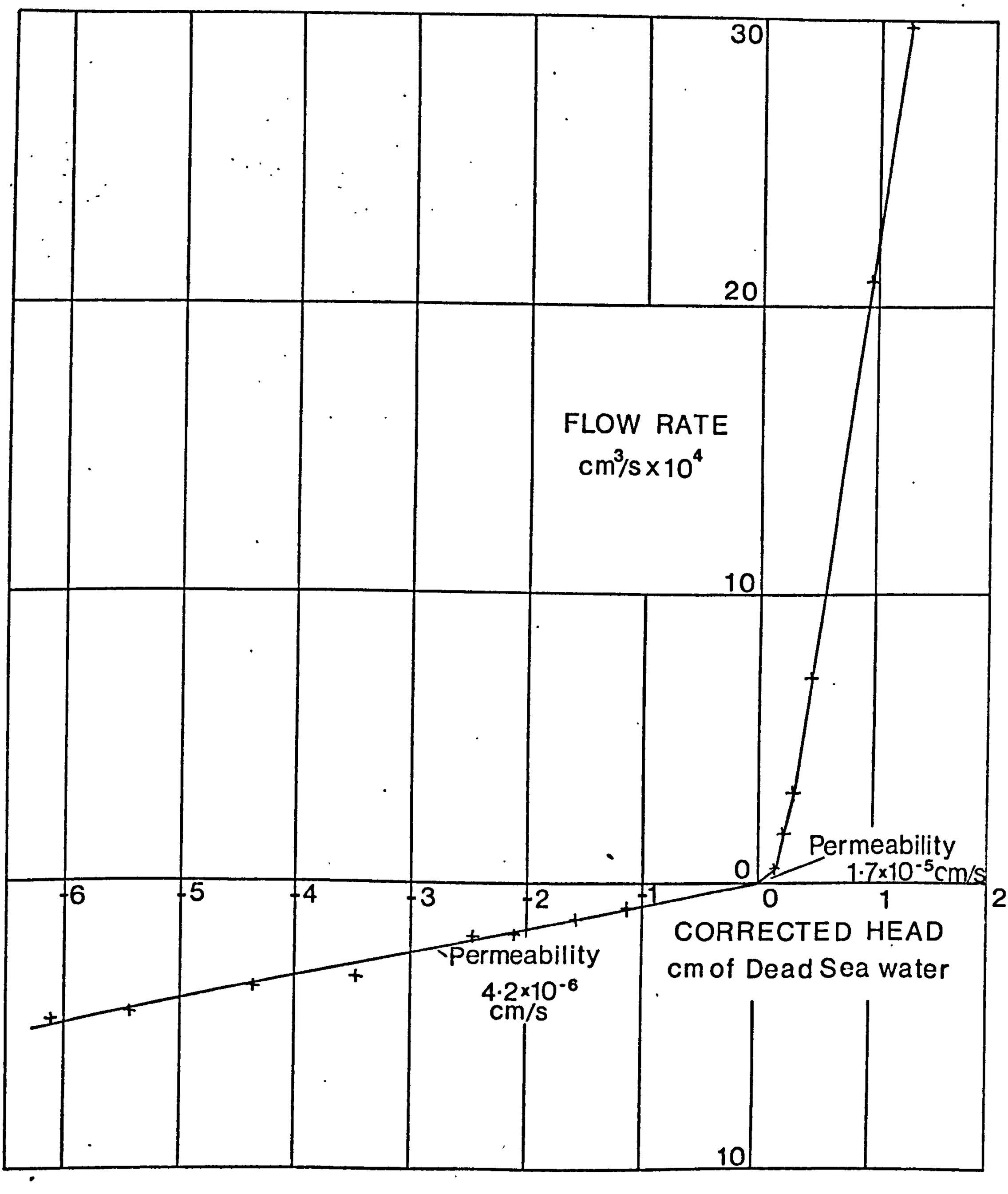


Fig. 48 PIEZOMETER TESTS WITH SLURRY AND NO GRAVEL

Failure was by piping to a point about 1½cm from the piezometer and it appears that even under these conditions the slurry does not flow.

The piezometer tests thus established that the slurry seals a gravel by forming a gel structure in the voids and that a very substantial permeability reduction can be achieved. However they gave no indication of the maximum hydraulic gradient a slurry/gravel combination could withstand. It was therefore decided to perform a series of conventional constant head permeability tests on slurry/gravel samples and to measure the failure gradients.

5.2 Constant head permeability tests

a. The apparatus and test procedure

The slurry/gravel samples were tested in the 3in diameter permeameters used for the ageing tests and as no consolidation measurements were required the hydraulic network was simplified to upstream and downstream reservoirs and the horizontal flow gauge (Fig.42). Pressure differences were measured from the reservoir levels and after some experimentation the kerosene in the flow gauge was replaced by iso-amyl alcohol as this reduced the pressure drop across the menisci in the gauge from 1.3cm (Equn.4.3) to 0.2cm thus much reducing the head correction.

A coarse porous stone made from sand and epoxy resin and sealed round the edge by a rubber band was placed in the bottom of the permeameter cell. The cell was then filled with about 7½cm of slurry/gravel mixture and a further ½cm of clean gravel placed on top to accommodate

any displaced slurry. The sample was capped by a second porous stone and loaded with the hydraulic piston to 4psi, to prevent any movement under the applied hydraulic gradients, and then allowed to age for twenty-four hours. At the end of this period permeability tests were run first at a pressure difference of 0.5cm and thereafter at increments of 0.5cm until the sample failed.

All the samples tested in this manner failed (under gradients of less than 0.3) by piping up the walls. This was probably caused by leakage round the bottom porous stone directing flow to the edges of the sample where there is clearly the most direct flow path. To prevent this the annular space between the walls of the cell and the stone was sealed with wax. This method proved satisfactory and in all subsequent tests failure was through the body of the material.

b. Test results

The results for the Dry Clay slurry in $\frac{1}{4}$-$\frac{3}{16}$in (6.35-4.76mm) gravel are shown in Fig.49. The head was increased in increments of 0.5cm and the permeability tests lasted one hour at the low heads and half an hour at the higher heads. The sample had an initial permeability of 6×10^{-6}cm/s and this increased to 12.4×10^{-6}cm/s immediately prior to failure which occurred at a gradient of 1.8. At two points during the tests the head was reduced and the permeability remeasured and it can be seen that increasing the head permanently increased the permeability. This permanent increase in permeability was probably due to cracking within the sample as a considerable network of cracks became visible just prior to failure.

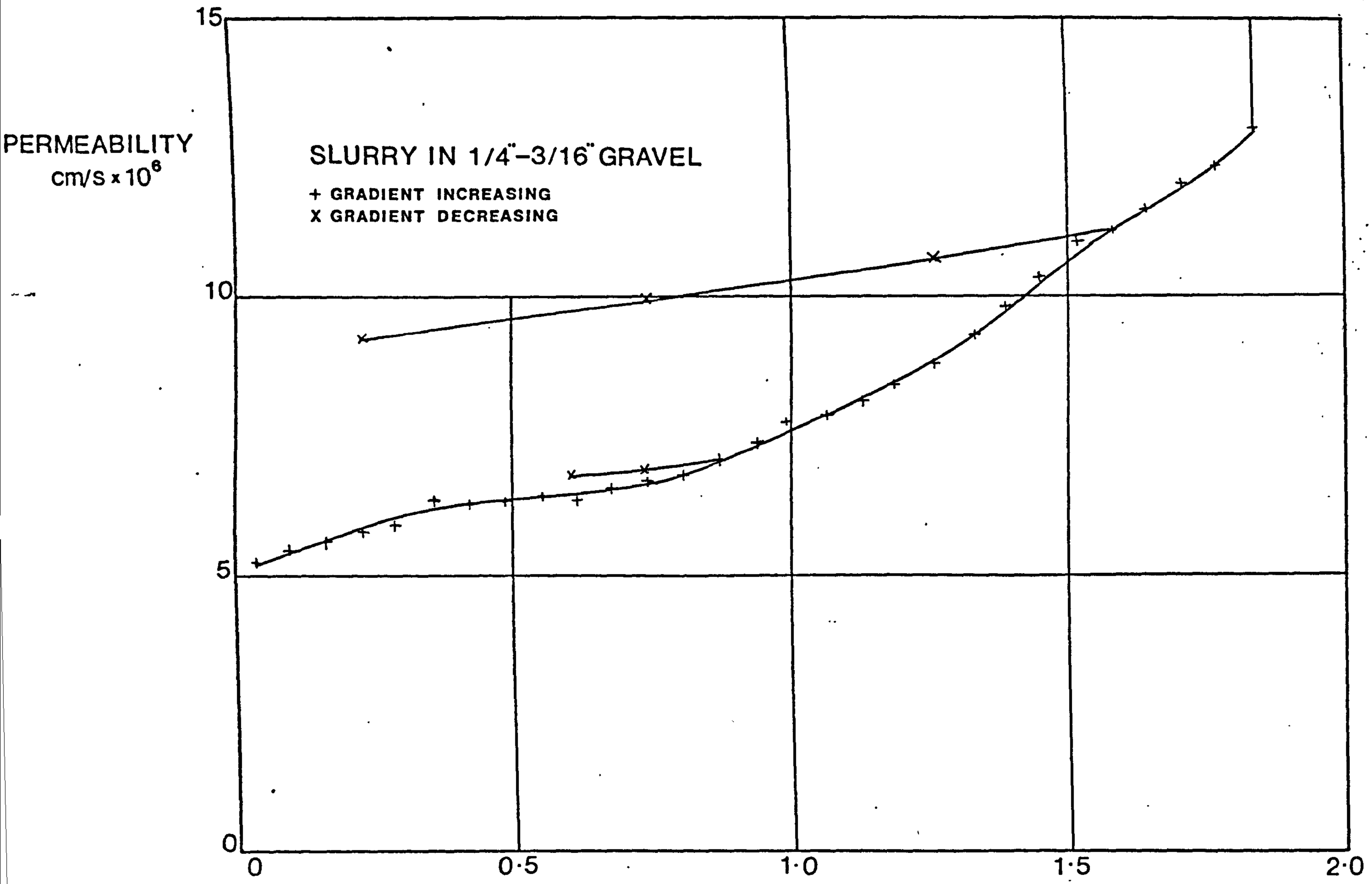

PERMEABILITY
cm/s x 10⁶
SLURRY IN 1/4"–3/16" GRAVEL
+ GRADIENT INCREASING
X GRADIENT DECREASING
15
10
5
0
0
0·5
1·0
1·5
2·0
HYDRAULIC GRADIENT

Fig. 49

Fig.50 shows the results for Dry Clay slurry in ¾-½ in (19.05-12.70mm) gravel. The initial permeability was 1.0 x 10^{-5}cm/s and the permeability just prior to failure 3.1x10^{-4}cm/s. Failure was by bulk movement of the slurry into the clean gravel at a gradient of 1.5. The head increment prior to failure was 3cm and it is probable that had this increase been made in smaller steps the sample would have failed at some intermediate head by cracking as in the previous tests.

Failure in these tests was taken as the point at which the permeability suddenly increased and was not based on the actual value of the permeability. In chapter 3 a permeability of 2x10^{-6}cm/s was suggested as the design permeability for the cut-off in a hydraulic structure. Thus on a permeability basis both samples would have been regarded as unsatisfactory, despite the very substantial permeability reduction achieved.

5.3 Bentonite slurries

a. Selection of slurry concentration

For the great majority of slurry trenching operations bentonite in fresh water slurries are used. It was therefore decided to test bentonite slurry/gravel mixtures to see what permeability reduction could be achieved.

An English bentonite was available for these tests. This clay is mined as a calcium bentonite and treated with sodium carbonate to produce the sodium form. Table XVIII shows the physical properties measured with the Fann V-G meter for 3-8% slurries of this bentonite twenty-four hours

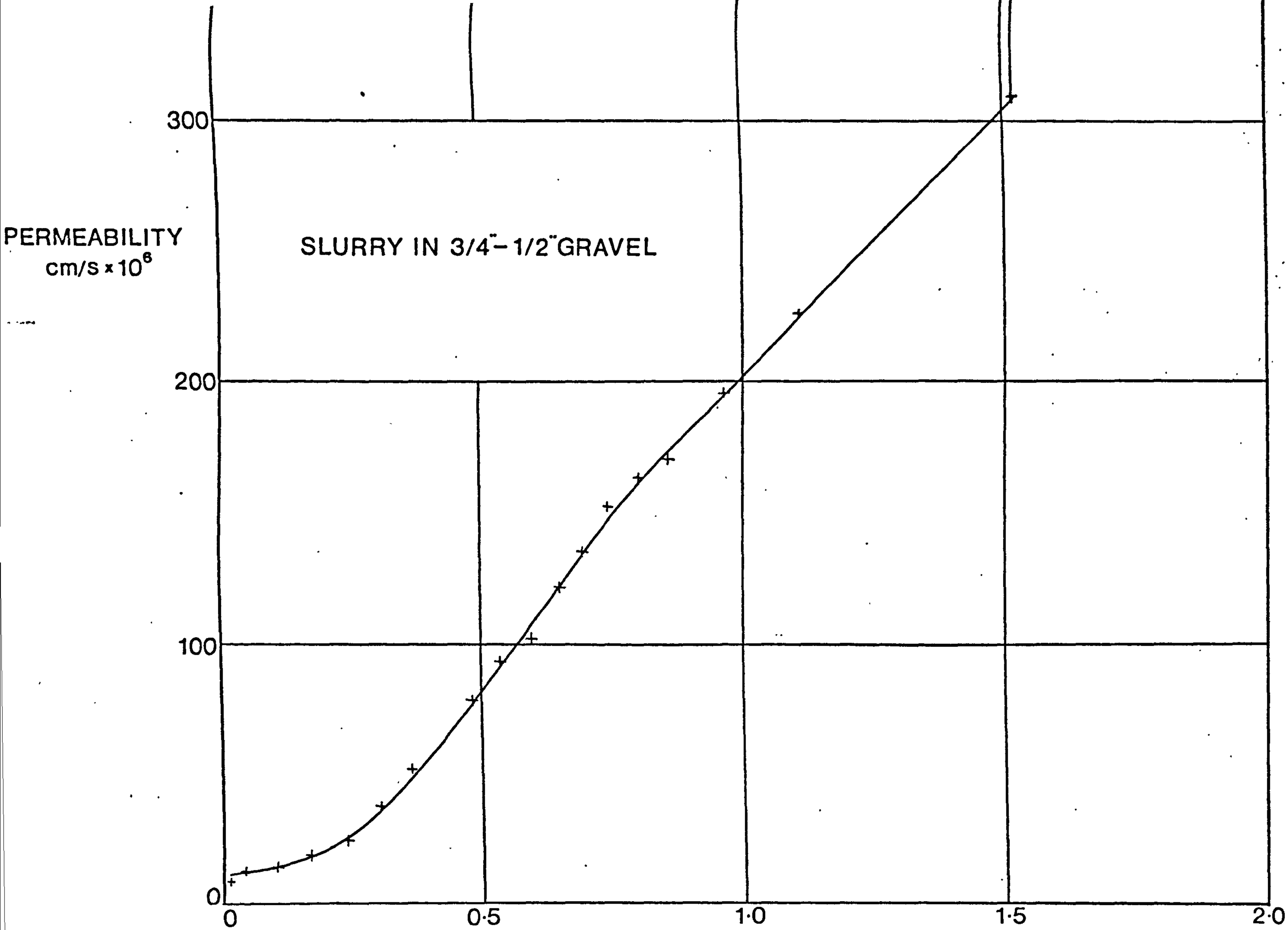

PERMEABILITY
cm/s x 10^6
SLURRY IN 3/4"–1/2"GRAVEL
300
200
100
0
0
0·5
1·0
1·5
2·0
HYDRAULIC GRADIENT

Fig.50

after mixing for five minutes in a high speed mixer.

Concentration grams/100 grams of slurry	Plastic Viscosity cP	Initial gel strength lb/100sq.ft	10 minute gel strength lb/100sq.ft
3	5.0	9	18
4	5.8	16.5	31
5	6.0	24.5	46
6	6.5	37	70
7	8.8	57	100
8	10.0	62	120

Table XVIII

For concentrations above 8% the Fann V-G meter readings become unreliable as the slurry gels so rapidly that it is difficult to ensure the annular space between the sleeve and bob of the instrument is filled with slurry. Above about 7% the slurry becomes difficult to pour without continuous stirring and a concentration of 6% was chosen as convenient for the bentonite/gravel tests. The tests were carried out in exactly the same manner as the previous tests on the Dead Sea slurries. Six close gradings of Thames ballast were tested with the 6% bentonite slurry.

b. Test Results

The results of the bentonite/gravel tests are summarised in Table XIX

Grading of gravel B.S.S.	Failure gradient	Initial permeability cm/sx10^6	Permeability just prior to failure cm/sx10^6	Gradient at which permeability exceeded $2x10^{-6}$ cm/s
$\frac{3}{4}$ - $\frac{1}{2}$in	3.4	0.12	16.7	1.6
$\frac{1}{2}$ - $\frac{3}{8}$in	5.4	0.10	3.0	4.8
$\frac{3}{8}$ - $\frac{1}{4}$in	7.9	0.18	7.0	5.5
$\frac{1}{4}$ - $\frac{3}{16}$in	11.7	0.10	1.2	11.7
$\frac{3}{16}$in-No.7	24.5	0.12	0.9	24.5
No.7-No.14	39.8	0.84	8.9	18.6

Table XIX

These permeability tests show that the bentonite/gravel mixtures can produce an adequate permeability reduction for hydraulic structures and that the slurry can withstand gradients greater than those normally encountered in such structures. The higher initial permeability observed in the No.7-No.14 sample may have been due to poor compaction of the gravel as it was difficult to sink the fine gravel in the bentonite gel, and when the piston was loaded some movement of the gravel may have occurred with consequent disruption of the slurry.

5.4 Prediction of failure gradient

If the failure is due to slurry being forced from the gravel voids the failure gradient can be derived by equating the applied forces to the maximum shear force the slurry can mobilise.

The derivation is very similar to that developed by Carman (1937) for the flow of fluids through a granular medium.

A granular medium can be regarded as a system of tortuous, interconnected channels in which the flow is not in general parallel to the direction of the applied hydraulic gradient. The effect of the tortuosity is to increase the length of the channels relative to the length of the medium and thus if the interconnections are ignored a channel can be represented as a straight tube inclined at an angle to the direction of the applied gradient.

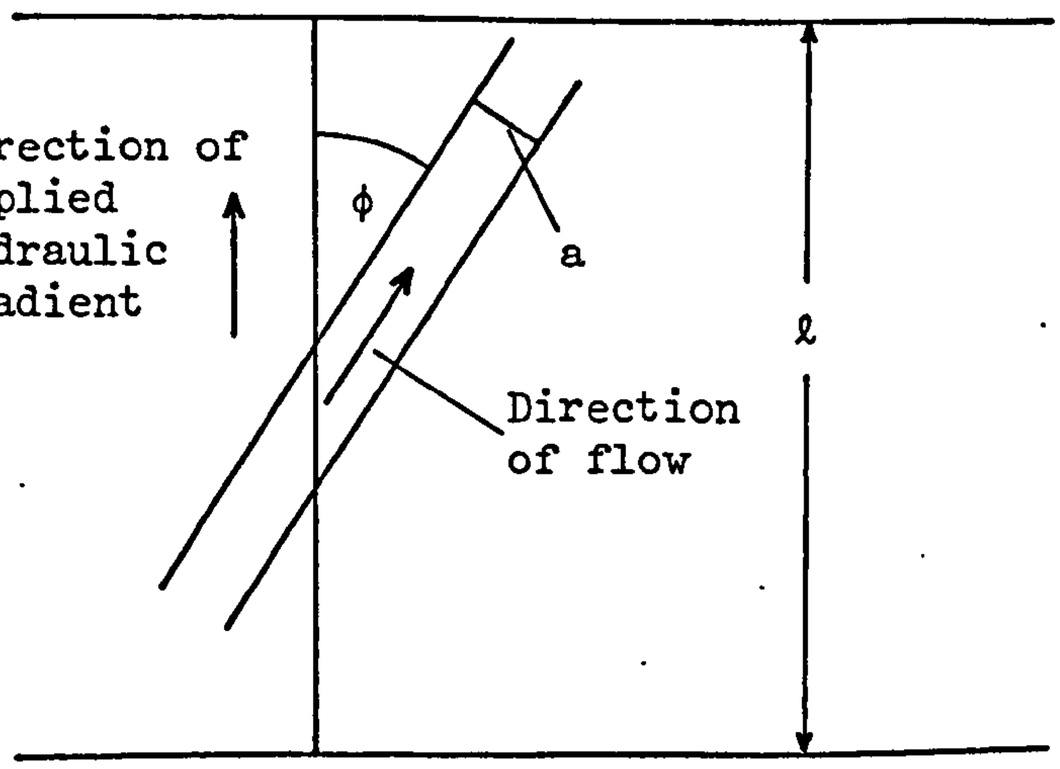


Figure 51

Figure 51 shows such a tube through a medium of length ℓ and it can be seen that the length of the tube is $\frac{\ell}{\cos\phi}$ and ϕ thus gives a measure of the tortuosity. If the tube is of area a then:-

$$\text{Tube volume} = \frac{a\,\ell}{\cos\phi}$$

For a cross-sectional area A of the medium the void volume is Aℓn, where n is the porosity

hence

$$\sum_{i=1}^{i=r} \frac{a_i \ell}{\cos\phi_i} = A\ell n$$

where r is the number of tubes in area A.

If these tubes can be represented as a single channel of area A_m inclined at an angle θ then:-

$$A_m = An\cos\theta$$

Thus if the medium consists of a sample of area A of a gravel/slurry mixture a hydraulic gradient i will produce a force tending to displace the slurry of:-

$i\gamma\ell.A\, n\cos\theta$ (fluid pressure x tube area)

where γ is the density of the permeating liquid.

Now if S is the surface area of the gravel per unit volume, the area of the channel walls in the sample will be AℓS.

The maximum force which the slurry can withstand

= Shear strength of slurry x area of channel walls

= $\tau.A\ell S$

where τ is the gel strength of the slurry.

There will also be a term involving the submerged weight of the slurry unless the slurry movement is horizontal.

$$\text{weight of slurry} = A\ell n\gamma'$$

where γ' is the submerged density of the slurry.

If this weight of slurry is to be moved at an angle of θ to the vertical the force required will be:-

$$A\ell n\gamma'\cos\theta$$

The maximum hydraulic gradient the slurry can withstand can be obtained by combining the three forces. For upward vertical flow of the permeating liquid:-

$$i\gamma\ell An\cos\theta = \tau A\ell S + A\ell n\gamma'\cos\theta$$

(for downward flow the slurry weight term will be negative)

where i is now the failure gradient for the slurry.

Hence
$$i = \frac{\tau S}{\gamma n\cos\theta} + \frac{\gamma'}{\gamma} \qquad 5.1$$

If the gravel particles are regarded as spheres of diameter equal to the mean particle diameter, D.

$$S = \pi D^2 N$$

Where N is the number of particles in unit volume

$$1-n = \frac{N\pi D^3}{6}$$

Hence $$S = \frac{6(1-n)}{D} \qquad 5.2$$

Substituting 5.2 into 5.1

$$i = \frac{6}{\cos\theta}\frac{\tau}{D\gamma}\frac{(1-n)}{n} + \frac{\gamma'}{\gamma} \qquad 5.3$$

Typical values of θ for viscous flow through granular materials are in the range 48°-51° (Carman 1938,9) giving values of $\frac{6}{\cos\theta}$ in the range 9-9.5.

For the permeameter samples the porosity n can be calculated from the weight of gravel placed and the thickness of the sample, γ' from the density of the slurry and if D is assumed to be the mean of the sieve sizes of the upper and lower limits of the gravel fraction, the gel strength of the slurry in the gravel voids, τ, remains the only unknown.

An estimate of τ can be made from the 10 minute gel strength measured with the Fann V-G meter, though the actual gel strength of the slurry will be greater after the ageing period in the permeameter. This gel strength could be measured by allowing the slurry to age in the viscometer but the instrument is not well suited for long term measurements as the sample is not sealed and tends to dry out during a test of more than about twelve hours duration and the bentonite slurries do not reach equilibrium strengths in such a time. If the gel strength in the

gravel is assumed to be a constant multiple (m) of the 10 minute gel strength, equation 5.3 becomes:

$$i = \frac{6}{\cos\theta} m \frac{\tau_{10}(1-n)}{D\gamma} \frac{}{n} + \frac{\gamma'}{\gamma}$$

where τ_{10} is the 10 minute gel strength.

Hence a plot of $(i\gamma-\gamma')$ against $\frac{\tau_{10}(1-n)}{D \quad n}$ should have zero intercept and slope $\frac{6m}{\cos\theta}$. Fig.52 shows such a plot for all the tests in the permeameters both on Dead Sea and bentonite slurries (The tests on attapulgite in Dead Sea water and Dry Clay slurry aged for six weeks are described in Chapter 6.) The results for the four coarser bentonite/Gravel tests and three of the Dead Sea tests lie very close to a straight line of zero intercept as predicted by the theory. It therefore appears that failure in these tests was by displacement of the slurry. Such a displacement was only observed when a large head increment was applied to the Dry Clay slurry in $\frac{3}{4}$-$\frac{1}{2}$in gravel sample and it thus seems likely that in general failure resulted from cracking produced by a very slight displacement. It was not expected that the results for the Dead Sea and bentonite samples should lie close to the same straight line as the values of m for the two materials might be different, but it appears that in practice they were very similar. It is significant that the Dry Clay slurry sample aged for six weeks failed at a higher gradient than would have been predicted from the other results. This may have been due to a greater increase in gel strength, that is a higher value of m achieved in the six weeks than in the twenty-four hours allowed for the other samples. The results

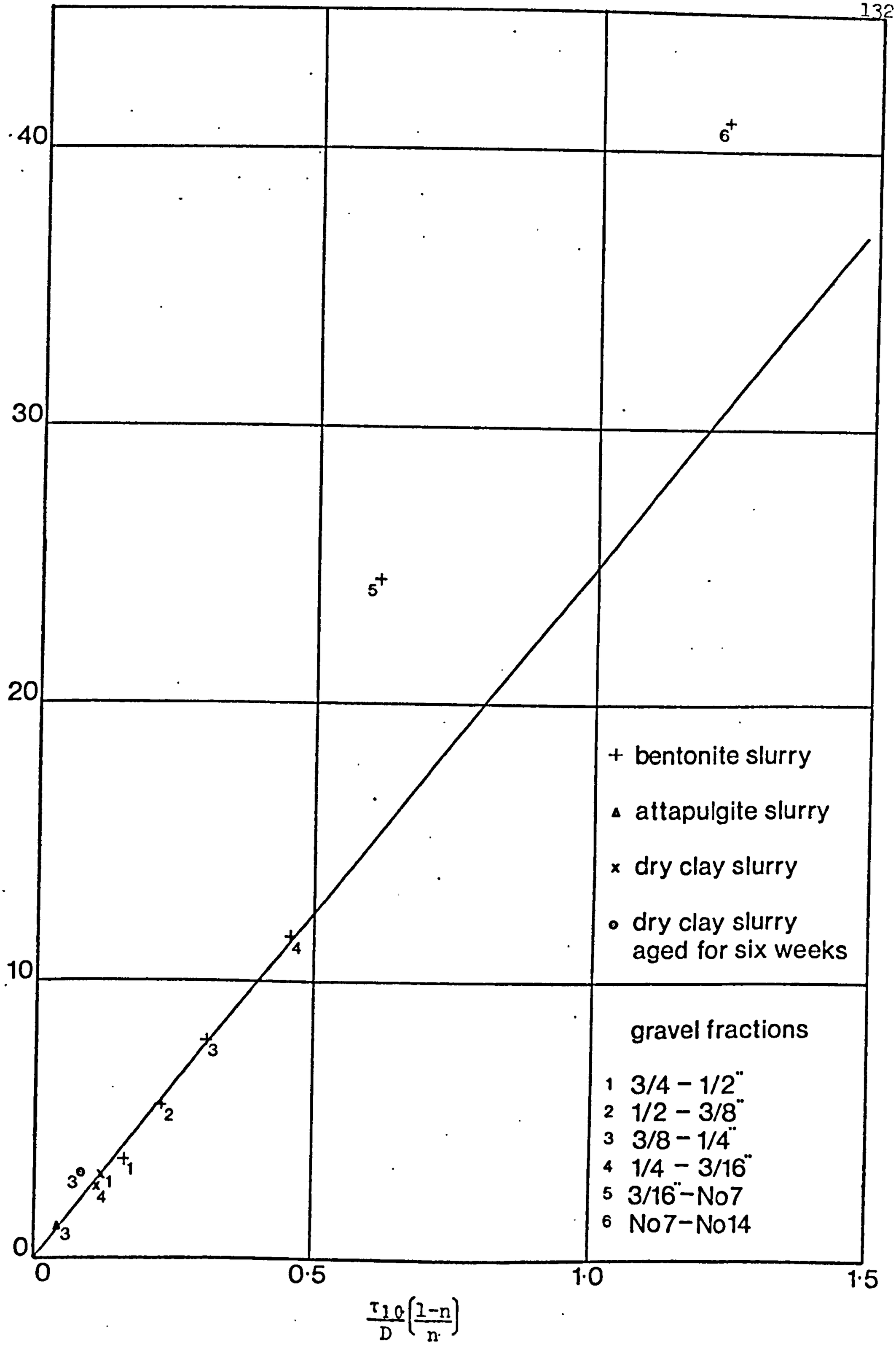


Fig. 52 RESULTS OF SLURRY/GRAVEL TESTS

for the bentonite in $\frac{3}{16}$in-No.7 and No.7-No.14 gravel are more surprising as failure occurred considerably above the expected gradient without any additional ageing. For the $\frac{3}{16}$in-No.7 gravel the failure was characteristic of a blow-out failure as the permeability suddenly increased from 3×10^{-7} to 10^{-3}cm/s; thus it appears that this gravel is better able to resist blowout than predicted by the theory. In the No.7-No.14 test no such sharp failure was observed, the permeability dropped steadily during the test to about 10^{-5}cm/s. When the gradient was held at 41 the permeability continued to drop and it appeared that failure was occurring by erosion of the slurry. The considerable increase in permeability during the test suggests that erosion was taking place at the lower gradients though the rate was too slow to be observed during a single test. The magnitude of the permeability increase and the slow rate at which it occurred suggest that erosion took place from cracks in the sample produced by the applied gradient and that the fine grading of the gravel prevented these cracks from spreading to produce failure as in the coarser gravels. If erosion had been from the downstream face only, as postulated for clays by Kassif and Zaslavsky (1965) a very considerable amount of material would have had to be removed to produce the observed permeability increase and no such loss occurred. Alternatively if the material was removed from one flow path only, the flow would have concentrated there with consequent much more rapid failure.

The slope of the line in Fig.52 is 25 hence:-

$$\frac{6m}{\cos\theta} = 25$$

thus m = 2.7 if $\frac{6}{\cos\theta}$ is assumed to be 9.25. Further information about the value of m under conditions similar to those in the permeameters can

be obtained by measuring the hydraulic gradient necessary to force a slurry of known 10 minute gel strength from a tube of uniform diameter. For this situation Equation 5.3 reduces to:-

$$i = 4m \frac{\tau_{10}}{D\gamma} + \frac{\gamma'}{\gamma} \qquad 5.4$$

A glass tube was therefore fitted with a porous stone at the base which was connected via a tap to a water pressure pot. The tube was then mounted vertically, the water level set at the top surface of the porous stone and the tap turned off. The tube was then part filled with bentonite slurry and the remainder filled with water and a second pressure pot connected to the top. The sample was allowed to age exactly as for the permeameter samples: the tap was then opened and the pressure across the sample increased in increments of 0.5cm, each increment being held for half an hour, until the slurry was seen to move. The shear strength of the slurry was calculated from Equation 5.4 and the results for two tube diameters are summarised in Table XX:

Tube diameter cm	10 minute gel strength N/m^2	Calculated gel strength N/m^2	m
0.7	59	101	1.71
1.1	73	132	1.81

Table XX

The average value of m from these tube tests is 1.76 which is somewhat lower than the 2.7 obtained from the permeameter tests. This may be due to a lower shear strength for bentonite against the smooth glass wall of the tube compared with that against the rougher gravel surfaces. Alternatively the value of m obtained from the permeameter results may be high as the value of $\frac{6}{\cos\theta}$ used in the calculations was measured for viscous flow of liquids and may well be higher for the displacement of a slurry.

CHAPTER SIX

THE BLEEDING OF SLURRIES

THE BLEEDING OF SLURRIES

6.1 Introduction

In the previous chapter no consideration was made of the permanence of the permeability reduction produced by a slurry in the voids of a gravel. The bentonite/fresh water slurries show little change with time but the Dead Sea slurries if allowed to settle in a beaker or tube show considerable bleeding; that is, the solids settle to leave a layer of free water at the surface. Fig.53 shows the solids level as a function of time for 14cm diameter tubes containing Dry Clay and Plastic Clay slurries. After one day there was more than 7% of free water in each tube and the bleeding appeared to continue indefinitely though at a decreasing rate. If this bleeding occurred in the slurry/gravel mixtures pockets of free water could develop in the voids of the gravel and if adjacent pockets connected, the gravel would no longer be sealed in a horizontal direction though the vertical permeability would be substantially unaffected unless the settlement was so severe as to allow vertical linkage between the water pockets.

6.2 Preliminary tests

To investigate this effect one of the permeameters was filled with Dry Clay slurry in $\frac{3}{8}$-$\frac{1}{4}$in gravel and then turned on its side so that the direction of flow was horizontal. The sample was allowed to age for six weeks and the permeability was then measured at a series of increasing heads until failure occurred. The results for this test

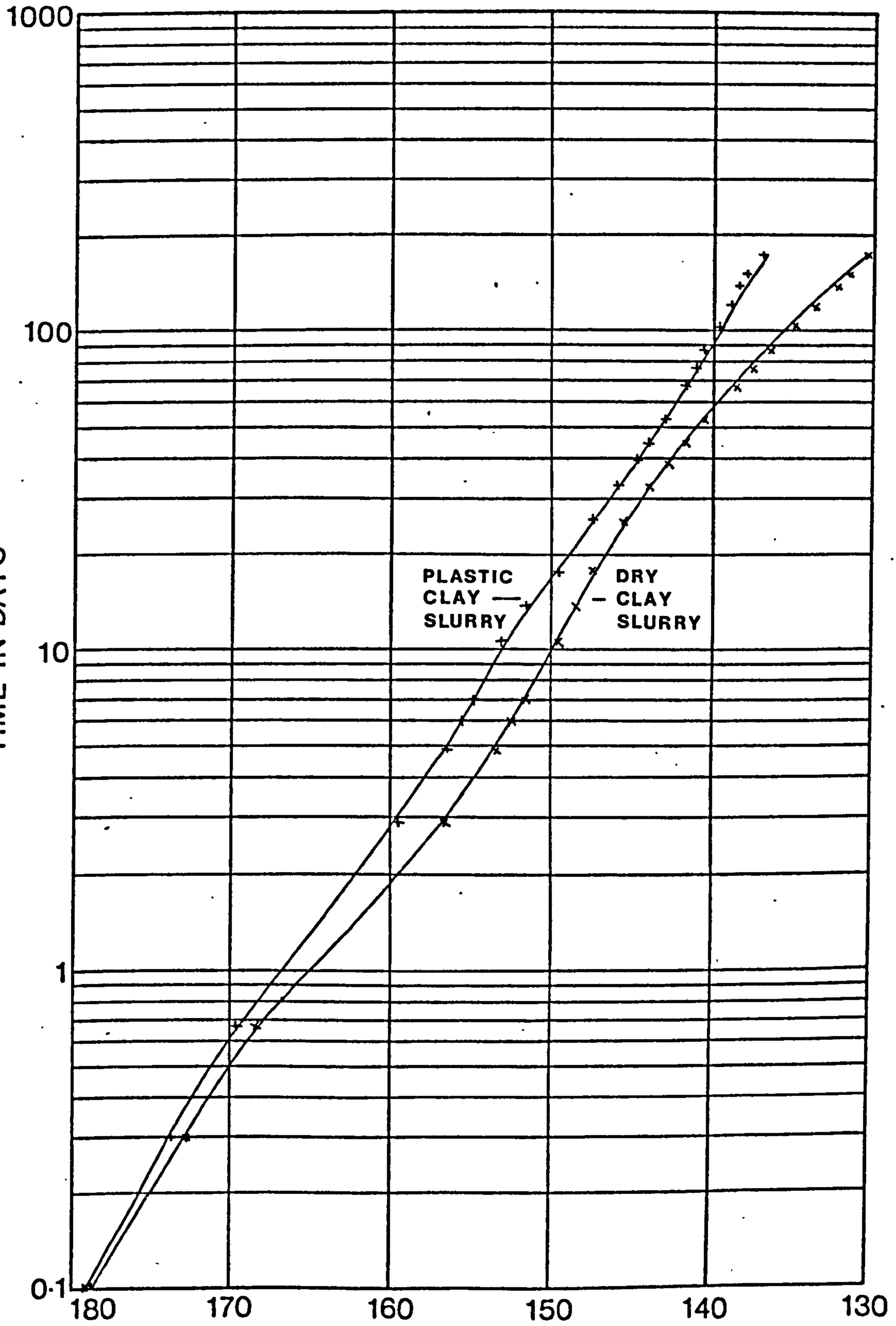


Fig.53 THE BLEEDING OF DEAD SEA SLURRIES

are shown in Fig.54 and it can be seen that the initial permeability ($4x10^{-6}$cm/s) and the failure gradient (2.2) are very similar to those obtained in the previous permeameter tests. This suggests that no bleeding occurred though the effects of settlement may have been masked by an overall improvement of the sealing properties of the slurry with time. Failure occurred along the top of the sample as might be expected regardless of bleeding as the effective stresses would be at a minimum there.

6.3 Horizontal and vertical permeability measurement

a. The Apparatus

To avoid the problem of variation of the slurry properties with time a box was built in which both horizontal and vertical permeability measurements could be made. Ideally such a box should have two sets of porous stones one set mounted horizontally and the other mounted vertically so that permeability tests can be made from A → B and from C ↓ D, but the stones for the tests C ↓ D will provide an easy flow path when testing A → B. To prevent this twelve small independent porous stones were used in each of the faces A, B, C and D of the box. These stones were made from sand and epoxy resin and were linked together by manifolds, in rows perpendicular to the two flow directions. Each face had 3 rows of stones and these could be connected together to make a test or isolated from each other when not in use. Fig.55 shows a diagram of the box and Fig.56 the manifold and flow diagram for one face. The manifolds of adjacent faces were

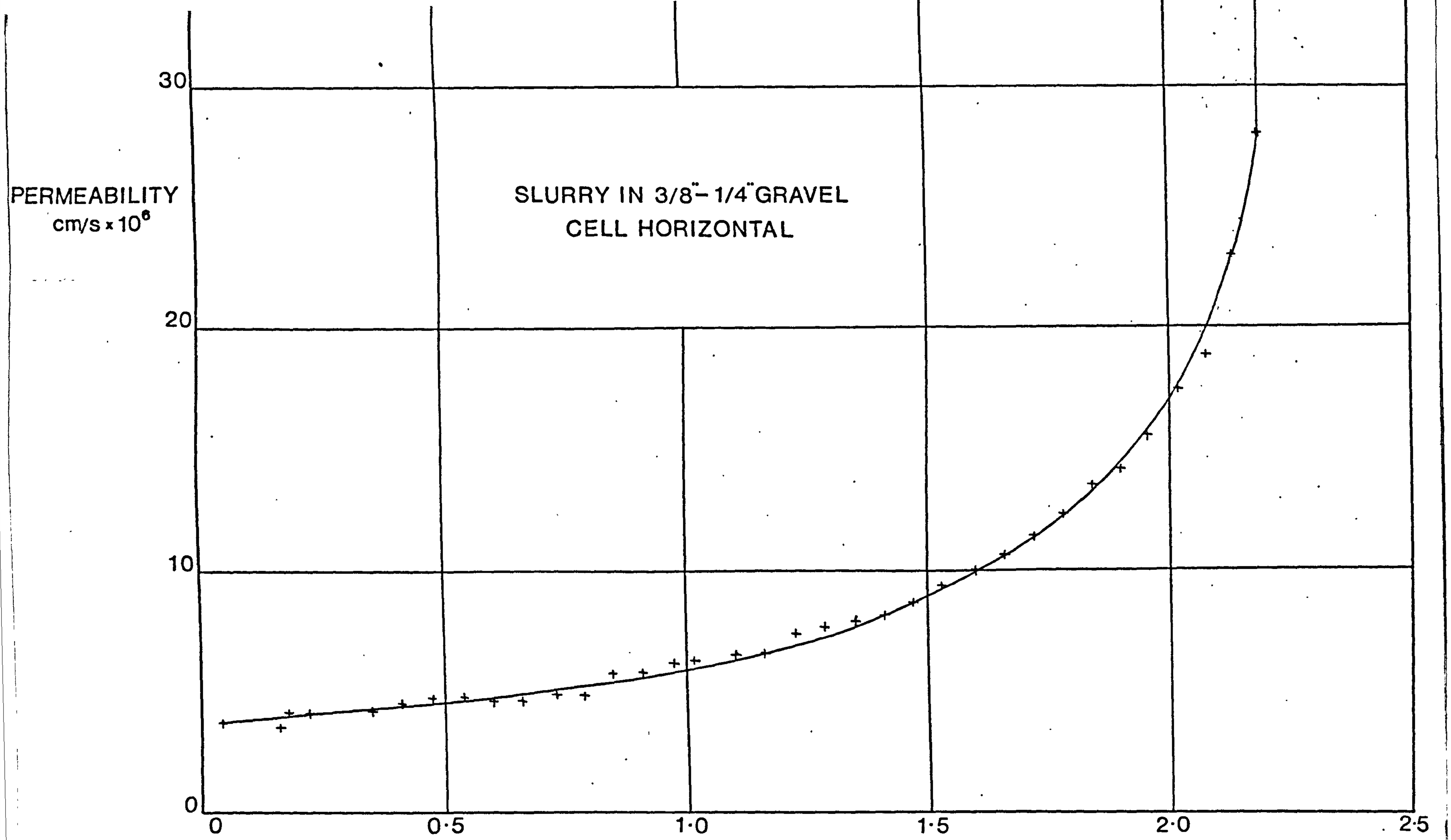
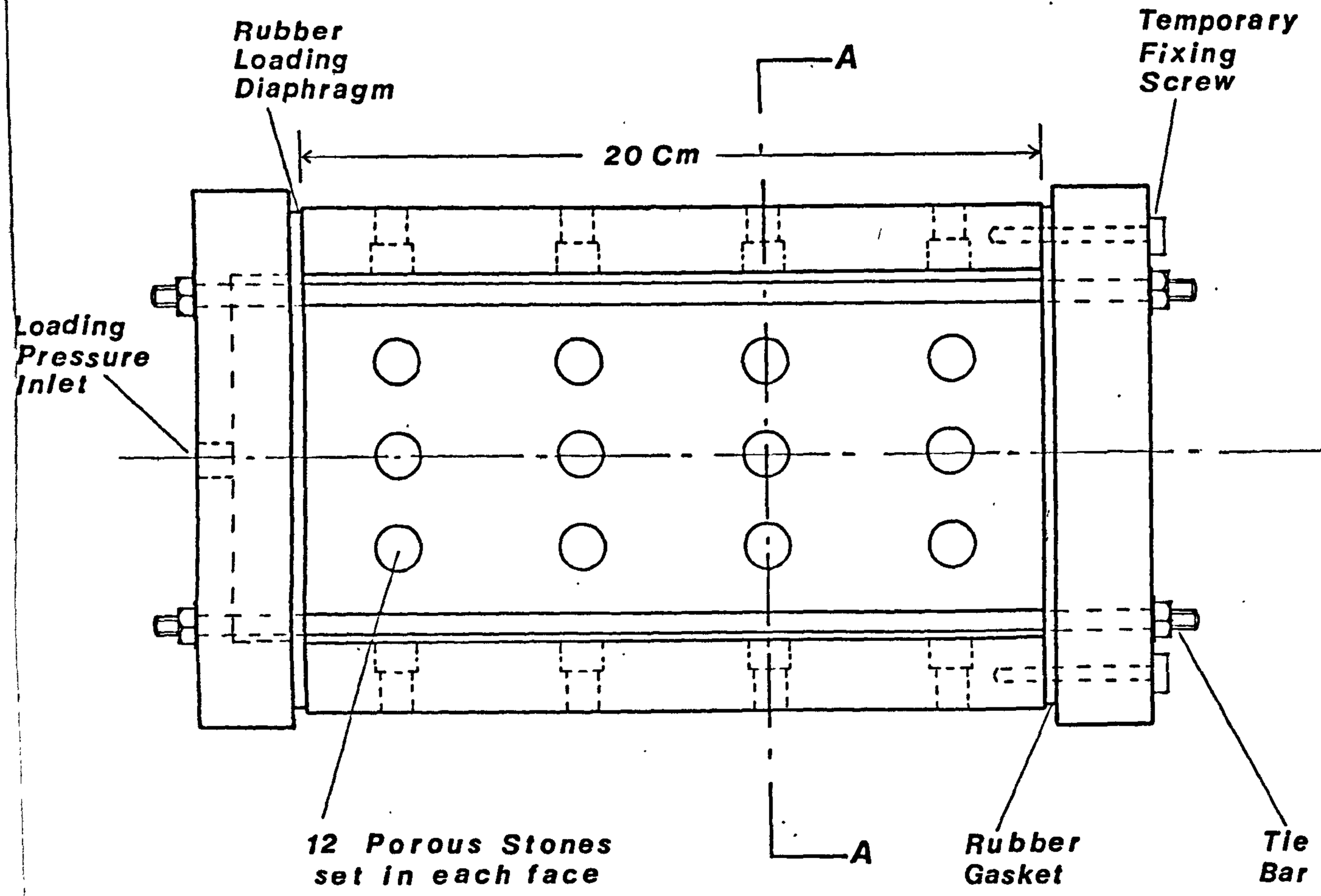
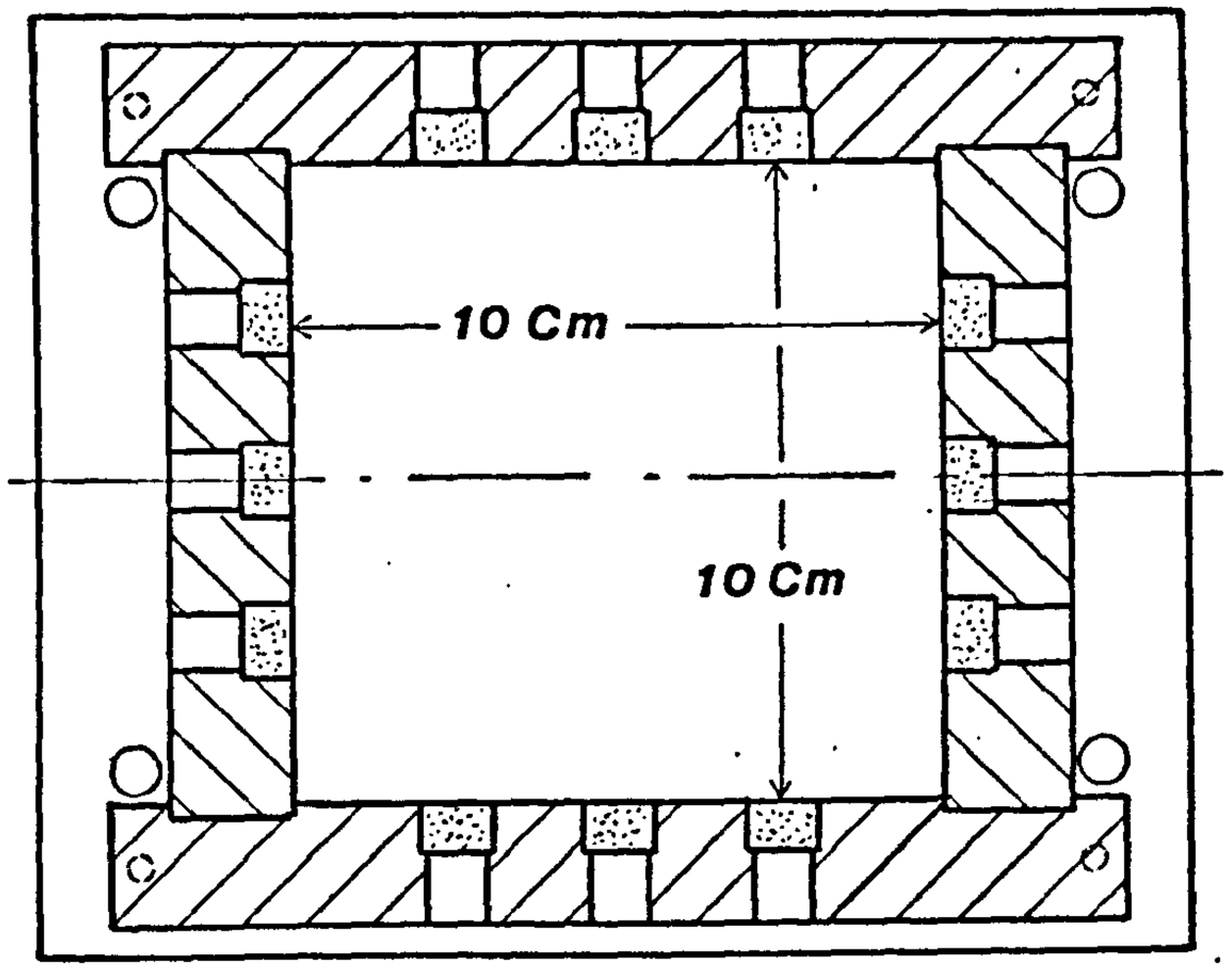

PERMEABILITY
cm/s x 10⁶
30
20
10
0
SLURRY IN 3/8"– 1/4" GRAVEL
CELL HORIZONTAL
0
0·5
1·0
1·5
2·0
2·5
HYDRAULIC GRADIENT

Fig. 54



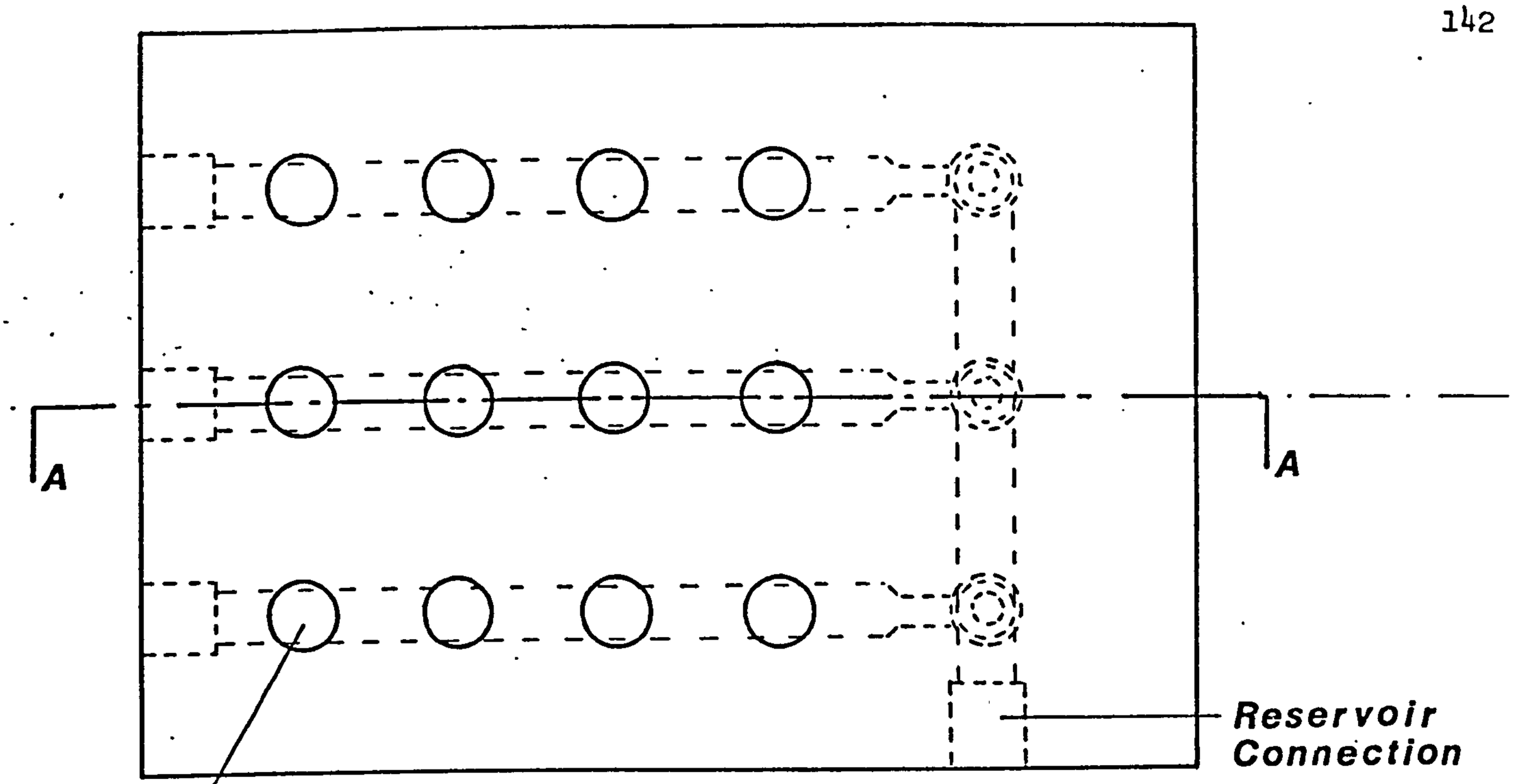
Plan of Box and Endplates

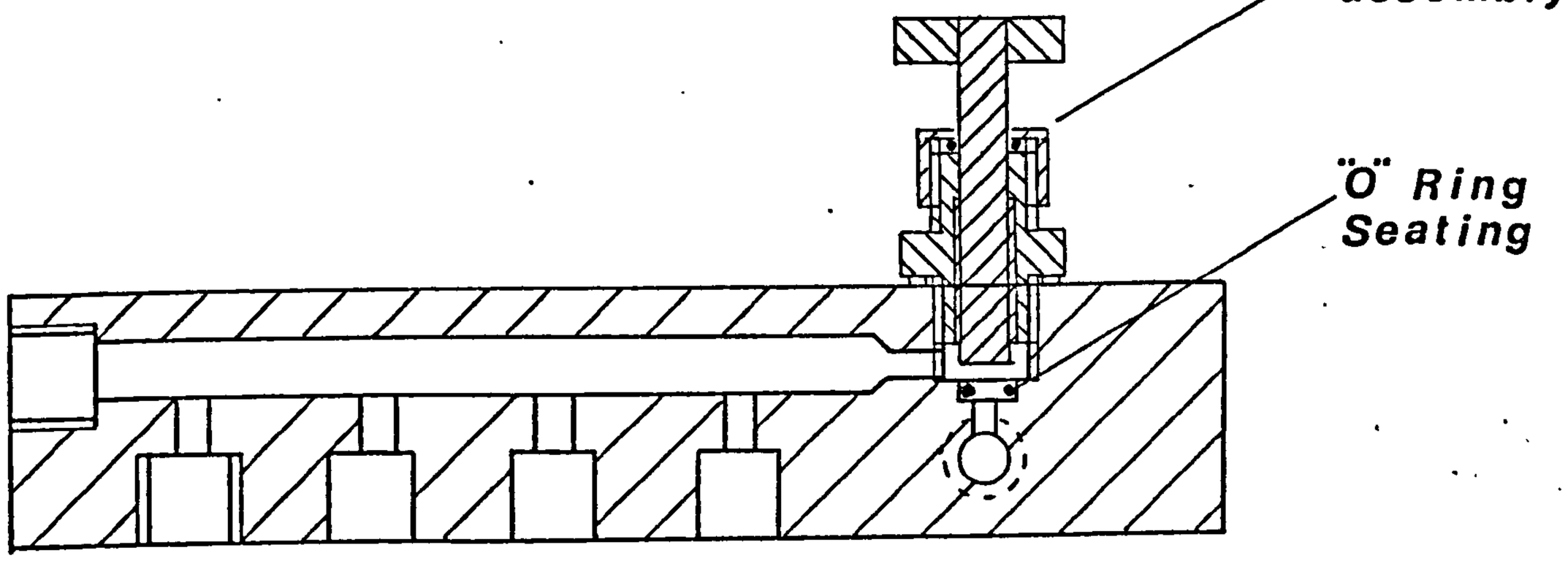


Section on AA

Fig.55 SLURRY TESTING BOX

Scale :- Half Full Size

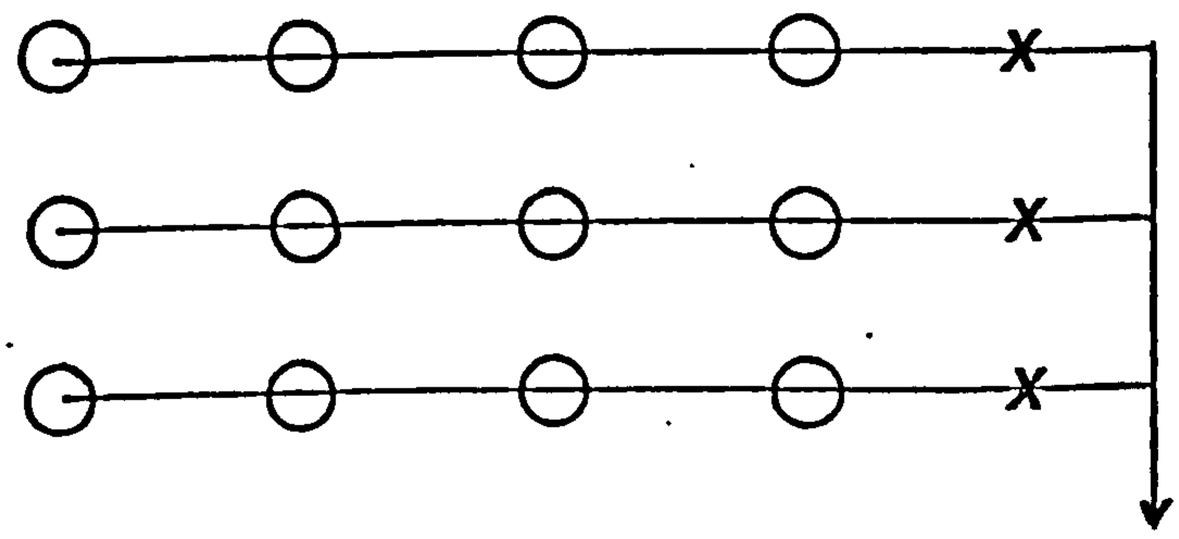




Section on AA

Scale:- Full Size

MANIFOLD FOR BOX



Flow Diagram for one Face

Fig. 56

connected together and thence to water reservoirs, one set directly and one set through the horizontal flow gauge. The box was made from Perspex so that the sample would be visible and across one end there was a rubber membrane against which hydraulic pressure could be applied to keep the gravel rigid and to seal any flow path across the end of the sample. Fig.57 shows a photograph of the assembled box and manifolds mounted on a slotted angle frame for ease of handling.

An electrical analogy was used to compare the effective area of the independent porous stones with that of ideal continuous stones. Brass discs were used to simulate the independent stones and copper sheets the continuous stones. The resistance of the cell filled with tap water was measured (with an A.C. bridge and oscilloscope) between opposite faces using first the brass discs and then the copper sheets. The ratio of resistances was 2.0:1; that is, the discs had an effective area of one half of that of the sheets. This result was confirmed by plotting equipotentials for the box, using the bridge and an electrical probe attached to a pantograph arm. Though the exact value of this area ratio is not important as it is intended to compare horizontal and vertical permeabilities and the ratio will be the same for each direction as the box is symmetrical.

b. <u>Filling the box</u>

The slotted angle frame was turned so that the loading axis of the box was vertical and the lower end plate held in position by the temporary fixing screws (Fig.55). The box was then filled with Dead Sea water and the porous stones and associated pipework deaired and the taps on the manifolds turned off. Slurry was then slowly run in

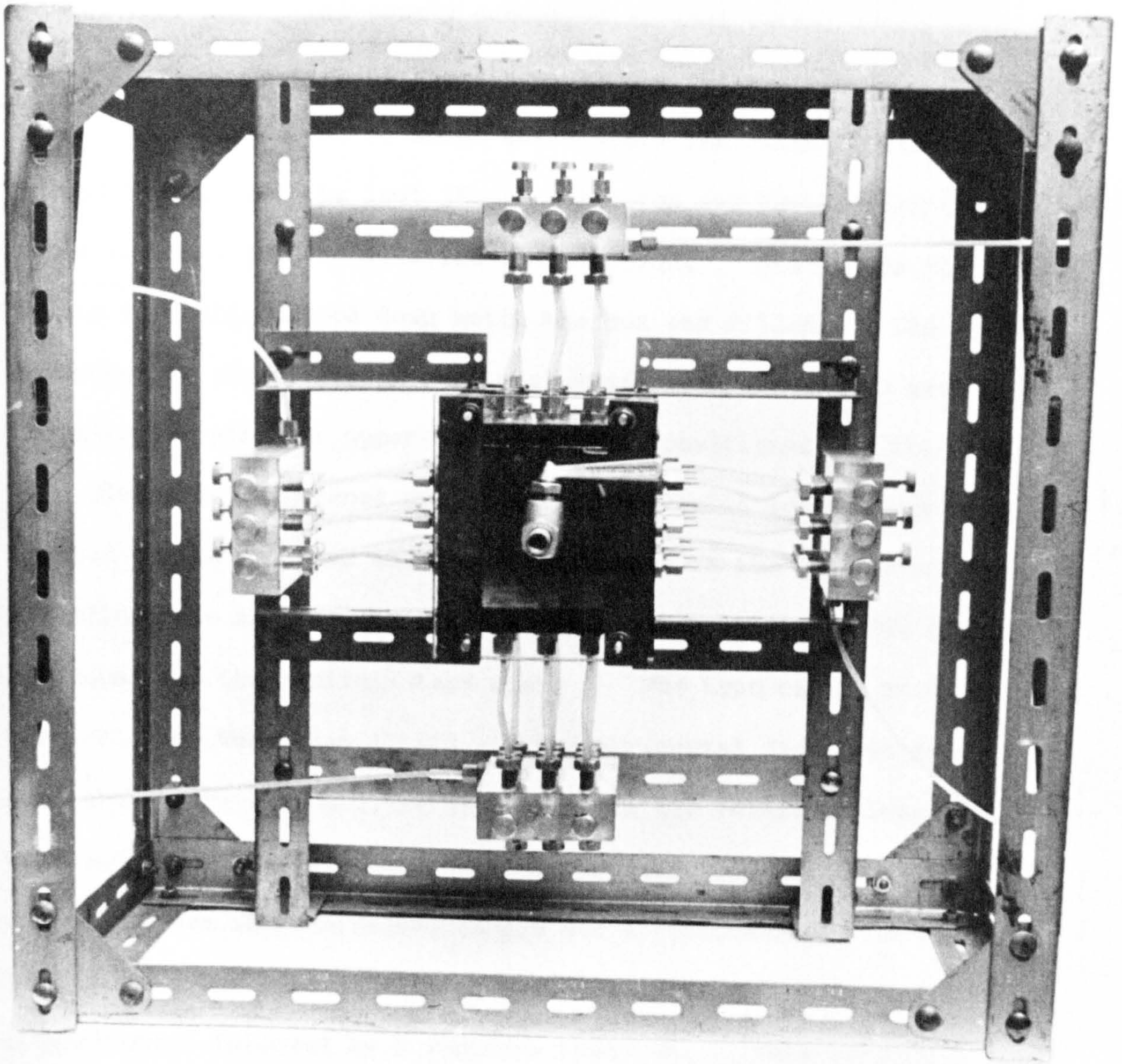


Fig. 57 THE SLURRY TESTING ASSEMBLY

from a pipe discharging at the bottom of the box and the displaced water and later slurry drawn off at the surface. By this means the porous stones were kept continuously submerged so that no air could enter. To avoid contamination of the slurry by the water originally present a volume of slurry equal to about three times the volume of the box was passed through and the last 350cm^3 collected and tested with the Fann V-G meter to check that no dilution had occurred. Gravel was then poured in and carefully tamped down until the box was filled. The loading membrane was placed gently on the slurry/gravel surface to avoid trapping any air, the upper end plate then positioned and the tie bars fitted and tightened up. A 4psi hydraulic loading was then applied to the membrane and the box returned to the horizontal position, and allowed to age for one week with the water levels in the reservoirs set equal and the manifold taps open. The taps on the manifolds for vertical flow were then closed and the horizontal flow rate measured for three hours with a 0.5cm difference in the reservoir levels. The taps were then changed and a similar vertical flow test run. The head difference was then increased to 1cm and a vertical and then an horizontal test run. The head difference was then increased a further 0.5cm and an horizontal followed by a vertical test run. This procedure was adopted to limit the number of tap changes required and was repeated at 0.5cm intervals until the sample failed.

Test results

The first sample tested was of Dry Clay slurry in $\frac{3}{8}$-$\frac{1}{4}$in Thames ballast and though the gradient was increased in increments to 10 no failure was observed. On stripping down the box the porous stones were found to be clogged with Dry Clay and the test results were therefore

disregarded. These porous stones were removed and replaced by coarser ones and for subsequent tests the slurries were made from attapulgite clay, all of which was finer than a No.200 B.S. sieve, with no additional native clay. Attapulgite slurries were used as they bleed rapidly because the needle like shape of the particles inhibits the formation of a structure.

5% and then 4% (by weight) attapulgite slurries in Dead Sea water were tested with $\frac{3}{8}$-$\frac{1}{4}$in Thames ballast, and a plot of corrected permeability (corrected for the effective area of the porous stones) against hydraulic gradient for these tests is given in Fig.58. For the 5% slurry there was no significant difference between the horizontal and vertical permeabilities and no bleeding was visible. Similarly the 4% slurry showed no bleeding but the initial horizontal and vertical permeabilities were slightly different at 11.4×10^{-6}cm/s and 8.2×10^{-6}cm/s respectively though this difference decreased as the gradients were increased possibly due to the mixing action of the changes in flow direction. This slight permeability difference would imply a very small amount of bleeding and could alternatively be explained if the attapulgite needles took up preferred orientations under the action of gravity; a phenomenon known to occur in dilute clay suspensions and used to prepare orientated samples for X-ray diffraction (Grim, 1962)

A 5% attapulgite slurry in $\frac{3}{8}$-$\frac{1}{4}$in gravel was also tested in one of the permeameters as a check on the operation of the box and the results are included in Fig.58 from which it can be seen that the permeabilities measured in the box are in good agreement with those from the permeameters.

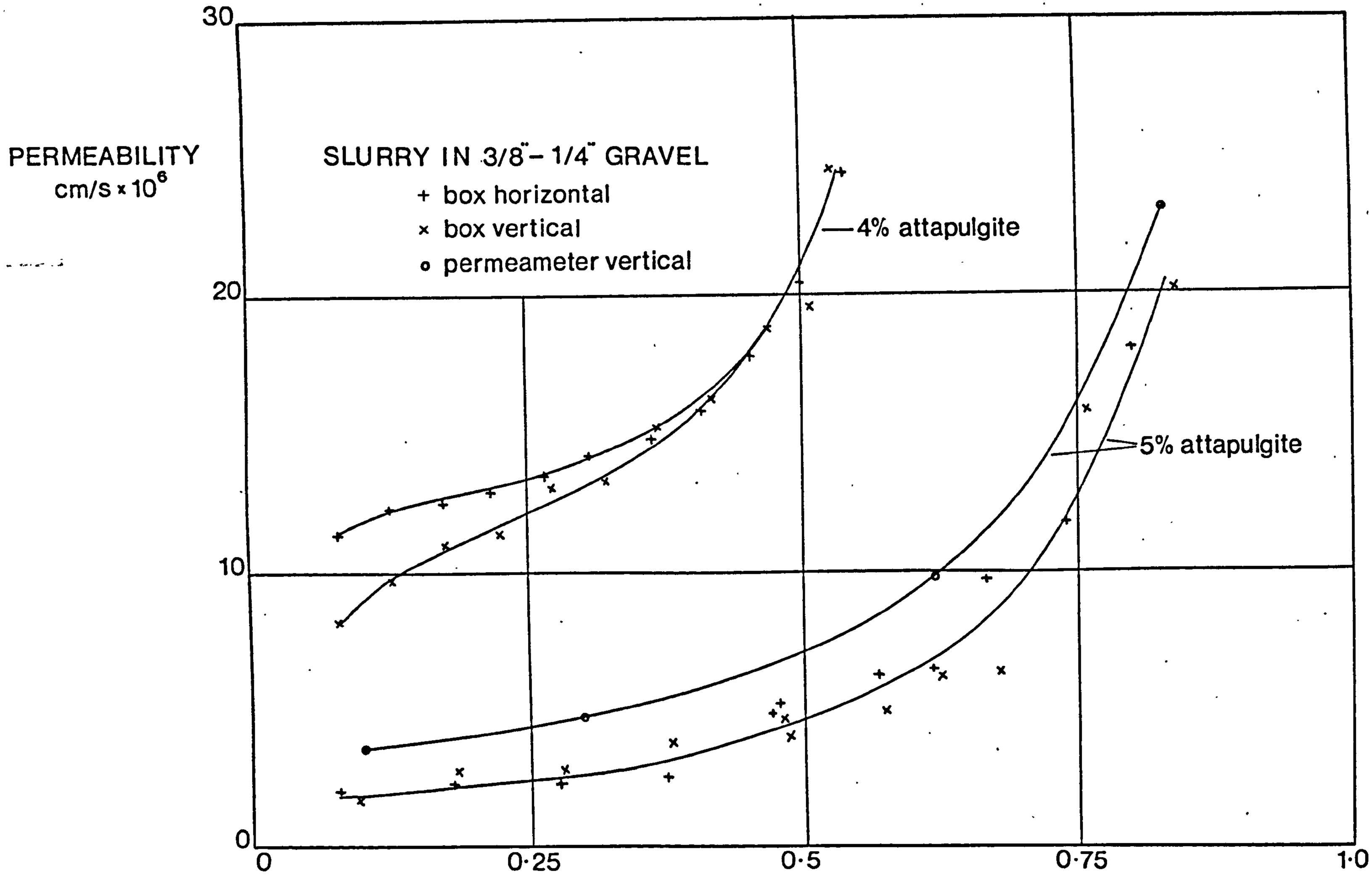

PERMEABILITY
cm/s × 10⁶
SLURRY IN 3/8"– 1/4" GRAVEL
+ box horizontal
× box vertical
o permeameter vertical
4% attapulgite
5% attapulgite
30
20
10
0
0
0·25
0·5
0·75
1·0
HYDRAULIC GRADIENT

Fig. 58

6.5 The effect of void size

The tests showed that the bleeding of a slurry is either prevented or severely limited in the voids of a gravel. To investigate this effect six glass tubes ranging in diameter from 0.2 to 1.5cm were sealed at one end, filled with Dry Clay slurry to a depth of 25cm, mounted vertically and capped with wax. The settlement of the slurry was then observed over a period of six months.

In the two smallest tubes (diameters 0.2 and 0.3cm) no bleeding was observed. The three intermediate tubes (0.4, 0.6, 0.8cm diameters) settled about 0.4cm over a period of two weeks and thereafter no bleeding was observed. The 1.5cm diameter tube continued to bleed for about 3 months though the total settlement in this time was only 0.8cm. It can be seen from these results and those for the 14cm diameter column shown in Fig.53 that the diameter of the tube markedly effects the bleeding of the slurry. This can be explained if the slurry solids are regarded as being supported from the walls of the tube by the gel strength of the slurry structure in much the same way as this gel strength enables slurry in a tube to withstand an applied hydraulic gradient.

Consider a column containing a slurry of uniform density and gel strength which has bled to produce a depth of pore fluid h as shown in Fig.59.

For a section at a depth z below the slurry surface the vertical total stress σ is given by

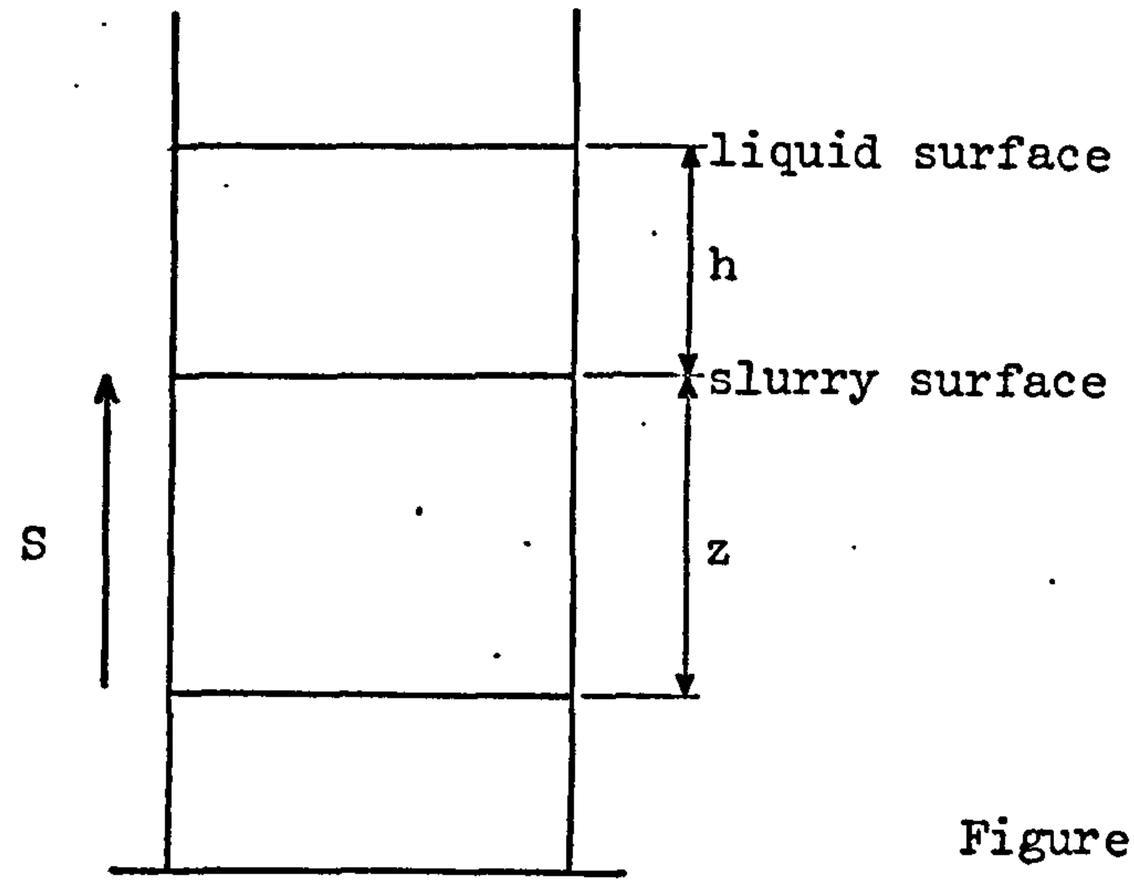


Figure 59

$$\sigma = \gamma_s z + \gamma_\ell h - S/A$$

where γ_s is the density of the slurry

γ_ℓ is the density of the pore fluid

A is the cross-sectional area of the column

S is the total shear force at the wall over depth z

Hence the vertical effective stress σ' at the section is:-

$$\sigma' = (\gamma_s - \gamma_\ell)z - \frac{S}{A} \qquad 6.1$$

If the slurry has a gel strength τ then:-

$$S = \pi dz\tau \qquad 6.2$$

where d is the diameter of the column.

Substituting equation 6.2 into 6.1 and rearranging yields:-

$$d = \frac{4\tau}{(\gamma_s - \gamma_\ell) - \sigma'/z} \qquad 6.3$$

If after placement of the slurry in the column the gel strength is sufficient for the slurry to be entirely supported by the shear force at the walls no bleeding should occur. Similarly if the gel strength reaches a sufficient value during bleeding no further settlement should occur. When the slurry is entirely supported at the walls the effective stresses will be zero throughout the column and equation 6.3 thus reduces to

$$d = \frac{4\tau}{\gamma_s - \gamma_\ell} \qquad 6.4$$

However if the effective stresses are not zero bleeding need not occur as the slurry may be able to withstand some stress without settlement. Thus equation 6.4 gives the largest diameter column in which bleeding should not occur and it may not occur in somewhat wider columns.

Substituting the properties of Dry Clay slurry into equation 6.4 yields:-

$$d = 2.2\text{cm}$$

Thus a column of Dry Clay slurry 2.2cm in diameter should not bleed. However bleeding of this slurry was observed in tubes of diameter 0.4cm and greater and thus this very simple theory does not fully explain the settlements observed.

6.6 The instrumentation of a column of slurry

To obtain information about the stresses in a column of slurry a 150cm high 10cm diameter Perspex tube was fitted with a base plate containing a central pressure transducer to measure total stress surrounded by an annular porous stone connected to a second transducer to measure the pore pressure. A third dummy transducer was provided to compensate for temperature changes. The transducers were commercially manufactured and had a gauge pressure range of 0-50psi (0-340kN/m^2) and a volume change of the order of 2.5×10^{-6} $in^3/lb/in^2$ (5.9×10^{-3} $mm^3/kN/m^2$). The three transducers were connected in parallel to a 10 volt stabilised power supply and the output voltages were measured with a digital voltmeter connected to an automatic data recording unit. This recording unit could take readings either continuously at ten a second or at preselected times from one minute to five hours and punch them directly on to paper tape. The apparatus was placed in a temperature controlled laboratory and the transducers were calibrated over the range 0-2 psi by filling the column to a series of water levels. With the column full of water the temperature in the laboratory was slowly altered from about $2^{\circ}C$ below to $2^{\circ}C$ above the normal operating temperature and the readings of the three transducers monitored, this process was then repeated at different water levels in the column. Hence as the reading of the dummy transducer was a function of temperature alone it could be used to correct the readings of the other two for temperature variations over the range of operating pressures. This temperature compensation was required even in the constant temperature laboratory as it was hoped to read the transducers to an accuracy of 1cm of water or 1/3500 of their full range. The digital voltmeter and recording gear were capable of distinguishing voltage changes corresponding

to pressure variations of 0.1cm of water.

The column was then filled with Dry Clay slurry which had been kept in the laboratory for the previous twenty-four hours to achieve temperature equilibrium. After filling the column the transducer readings were taken at one minute intervals for the first twenty-four hours and thereafter at two hour intervals for fourteen days. Fig.60 shows a plot of the measured pore pressure and total stress for the first seven days. During the first $1\frac{1}{2}$ days the results were as expected. The pore pressure dropped uniformly towards the theoretical minimum pore pressure, that is, the pressure produced by an equivalent column of pore fluid. The total stress dropped by about 2.5cm of water corresponding to a shear stress build up at the walls of about $6N/m^2$ compared with a 10 minute value for the slurry obtained with the Fann V-G Meter of $9N/m^2$. During the succeeding days however the pore pressure oscillated considerably and the total stress steadily increased above the theoretical maximum. It was therefore concluded that the transducers were not sufficiently stable for prolonged measurements of very small pressure differences and the test was abandoned. However if the results of the first $1\frac{1}{2}$ days can be accepted it appears that the transition of the slurry from liquid to solid was a gradual process which was almost completed in this time, and that the build up of shear stress at the walls was rather less than might be expected from the readings of the Fann V-G meter.

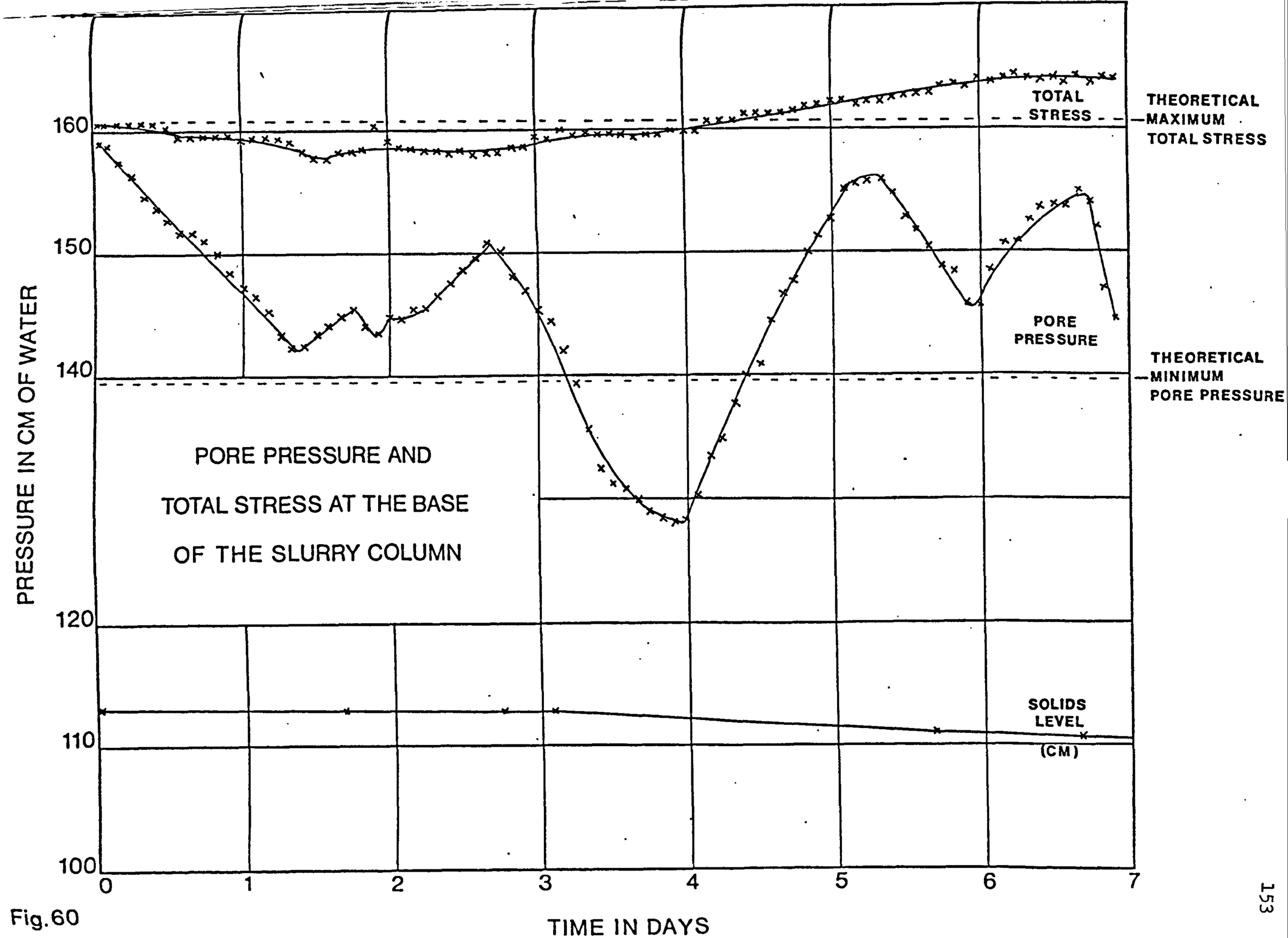

PRESSURE IN CM OF WATER
160
150
140
120
110
100
0
1
2
3
4
5
6
7
TIME IN DAYS
PORE PRESSURE AND
TOTAL STRESS AT THE BASE
OF THE SLURRY COLUMN
TOTAL
STRESS
THEORETICAL
MAXIMUM
TOTAL STRESS
PORE
PRESSURE
THEORETICAL
MINIMUM
PORE PRESSURE
SOLIDS
LEVEL
(CM)

Fig.60

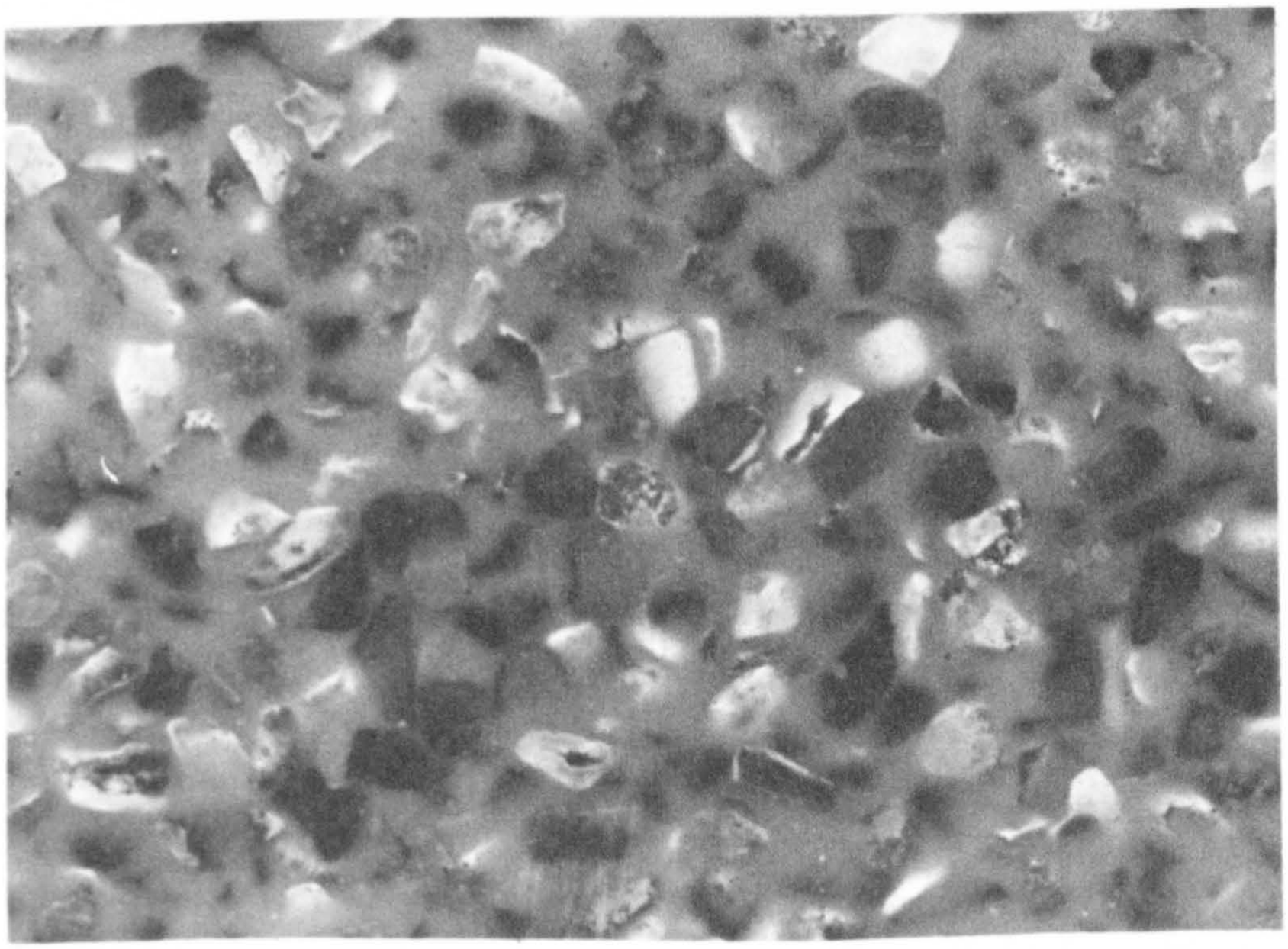
6.7 The permeability of a slurry/gravel mixture that bleeds

The tube tests show that gel strength has a profound effect on the bleeding of a slurry, and thus provide a basis for the design of a slurry/gravel combination that will bleed. The simple theory summarised in equation 6.4 demonstrates that for maximum bleeding a slurry must have a low gel strength and high submerged density. Such a slurry can be produced by mixing bentonite with a concentrated salt solution. The salt ensures that the bentonite remains aggregated so that no gel structure can form and the high submerged density can be achieved by using a high proportion of bentonite. An 18% suspension of bentonite in Dead Sea water was therefore made up and had the following properties:-

Submerged density	0.12 g/cm^3
Initial gel strength	4 lb/100sq.ft
10 minute gel strength	5 lb/100sq.ft

This slurry showed considerable bleeding in $\frac{3}{8}$-$\frac{1}{4}$in gravel. Fig.61a shows a photograph of this bleeding and Fig.61b shows a 6% bentonite/fresh water suspension in a similar gravel for comparison. Having thus obtained a slurry/gravel mixture that would bleed it was decided to measure the horizontal and vertical permeabilities but before setting up the box a preliminary test was made in one of the permeameters. The slurry/gravel mixture was tested under a vertical gradient of 0.05 and showed a permeability of 5×10^{-3}cm/s. Permeabilities of this order cannot be accurately measured either in the box or in the permeameters as a slight clogging of the porous stones could produce

(a) 18% Bentonite in Dead Sea water



(b) 6% Bentonite in fresh water

Fig. 61 Bentonite slurries in 3/8 – 1/4 gravel

such a permeability irrespective of the sample, and there was therefore no purpose in testing this slurry in the box. Although the measured permeability of 5×10^{-3} cm/s is considerably lower than that of the clean gravel the slurry cannot be regarded as having sealed the gravel and it must therefore be concluded that if a slurry has insufficient structure to gel in a void it has insufficient structure to seal that void.

CHAPTER SEVEN

DISCUSSION AND CONCLUSIONS

CHAPTER SEVEN

DISCUSSION AND CONCLUSIONS

From Chapters 1 and 2 it can be seen that a very wide range of slurry properties can be obtained by the use of chemical treating agents. In drilling, considerable use is made of these chemicals and muds are often tailored to individual borehole conditions. In slurry trench work the properties required are much less critical. The density required for the slurry can be calculated from formulae such as those developed by Nash and Jones (1963) but no definitive basis is available for the choice of viscosities and gel strengths, and slurries must be designed empirically to suit digging and backfilling conditions. For digging in some clays water could probably be used in place of a slurry if the density was sufficient and excessive softening of the formation did not occur. If concrete is to be placed the slurry must have low viscosity and gel strength (perhaps a plastic viscosity of about 15cP and a 10 minute gel strength of less than 20lb/100sq.ft) to avoid pockets of slurry being trapped in the concrete. For soil backfills a thicker slurry may be advantageous for although this will inhibit the settling of the fill the presence of the slurry will improve the permeability even of a clay backfill, especially if the fill is similar to the Asphaltic Clay, that is, it does not readily breakdown and behaves rather like a well graded sand. It should however be noted that the permeability of a clay/slurry mixture is greatly dependent on the degree of consolidation. For example the permeability measured

in the 3in diameter permeameters for Dry Clay in Dry Clay slurry was about 1.2×10^{-7}cm/s under an 8psi overburden pressure but for the same material in the large tank consolidated only by self weight the permeability was of the order of 4×10^{-6}cm/s. In a trench the degree of consolidation may be limited by arching and it is therefore necessary to carry out tests under low overburden pressures when selecting backfill materials, except for cut-offs involving well graded gravel/sand/clay backfills as these show little variation of permeability with overburden pressure.

The slurry in gravel tests show that if a slurry is used a very substantial permeability reduction can be achieved even with a coarse backfill provided that the voids are small enough to prevent the slurry being blown out. For the bentonite slurries the permeability under low gradients was about 10^{-7}cm/s rising to about 10^{-5}-10^{-6}cm/s just prior to failure. Even with the $\frac{3}{4}$-$\frac{1}{2}$in gravel the slurry had a permeability of less than 2×10^{-6}cm/s for gradients up to 1.6 and such a combination could therefore provide a satisfactory cut-off for many hydraulic structures. The gradient at which a slurry will fail in a gravel can be predicted from the results shown in Fig.52 but for most slurry/gravel combinations this gradient will be considerably greater than any encountered in practice.

The horizontal and vertical permeability tests described in Chapter 6 show that the settlement of a slurry is greatly restricted by a gravel and that a permanent horizontal as well as vertical permeability reduction can be achieved. This reduction of permeability has been used to obtain dry workings at a gravel pit (cement, lime and gravel, 1964). A 5%-6% bentonite suspension was used with a well graded gravel backfill

to which an additional 2% of pulverised fly-ash had been added to improve the fines content. This cut-off performed satisfactorily and the results described in Chapter 5 would suggest that the addition of pulverised fly-ash was unnecessary.

The results of the slurry settlement tests in the tubes provide a basis for the design of slurries so that bleeding will not occur and in conjunction with the results of Chapter 5 a slurry can therefore be specified to seal any gravel.

APPENDIX 1

The properties of Dead Sea water

The composition of Dead Sea water varies with the time of year and also the location in the Sea (Neev and Emery, 1967). The figures quoted below are those measured for the water used in the test programmes

a Physical properties:

Density at 20°C	1.226 g/cm^3
Viscosity at 20°C	2.80 cP
Boiling point	109°C
pH	7.1

b Chemical composition:

The concentrations of the major constituents are listed below in grams/litre.

Sodium	46.7
Potassium	7.9
Magnesium	36.4
Calcium	14.2
Chlorine	209.7
Bromine	4.5
Bicarbonate	0.5
Sulphate	0.5

APPENDIX 2

A note on the measurement of water content of Dead Sea soils

For soils containing salt water the measured water content will be different from the actual water content. For example, if we take 100 g of dry soil containing no salts and add to it 36 g of Dead Sea water, the actual water content is 36% but if the wet soil is dried in an oven only (say) 25 g of water will be driven off leaving 11 g of salt, and the dried sample will weigh 111 g which leads to an apparent moisture content of 22.5%.

The problem then is to relate the weight of water driven off to the initial weight of salt water present. The main salts present in Dead Sea water are the chlorides of sodium, potassium, magnesium and calcium all of which are extensively hydrated, that is have molecules of water chemically bound to them. On drying samples containing such salts the free water is driven off fairly rapidly, depending on the surface area and thickness of the sample, but the bound water is driven off slowly and the amount driven off is a function of the drying temperature. Fig.A1 shows the variation of the measured water content of a sample of Plastic Clay containing Dead Sea water, with time and temperature. The free water is driven off during the first heating stage and the bound water is driven off in increments as the drying temperature is increased.

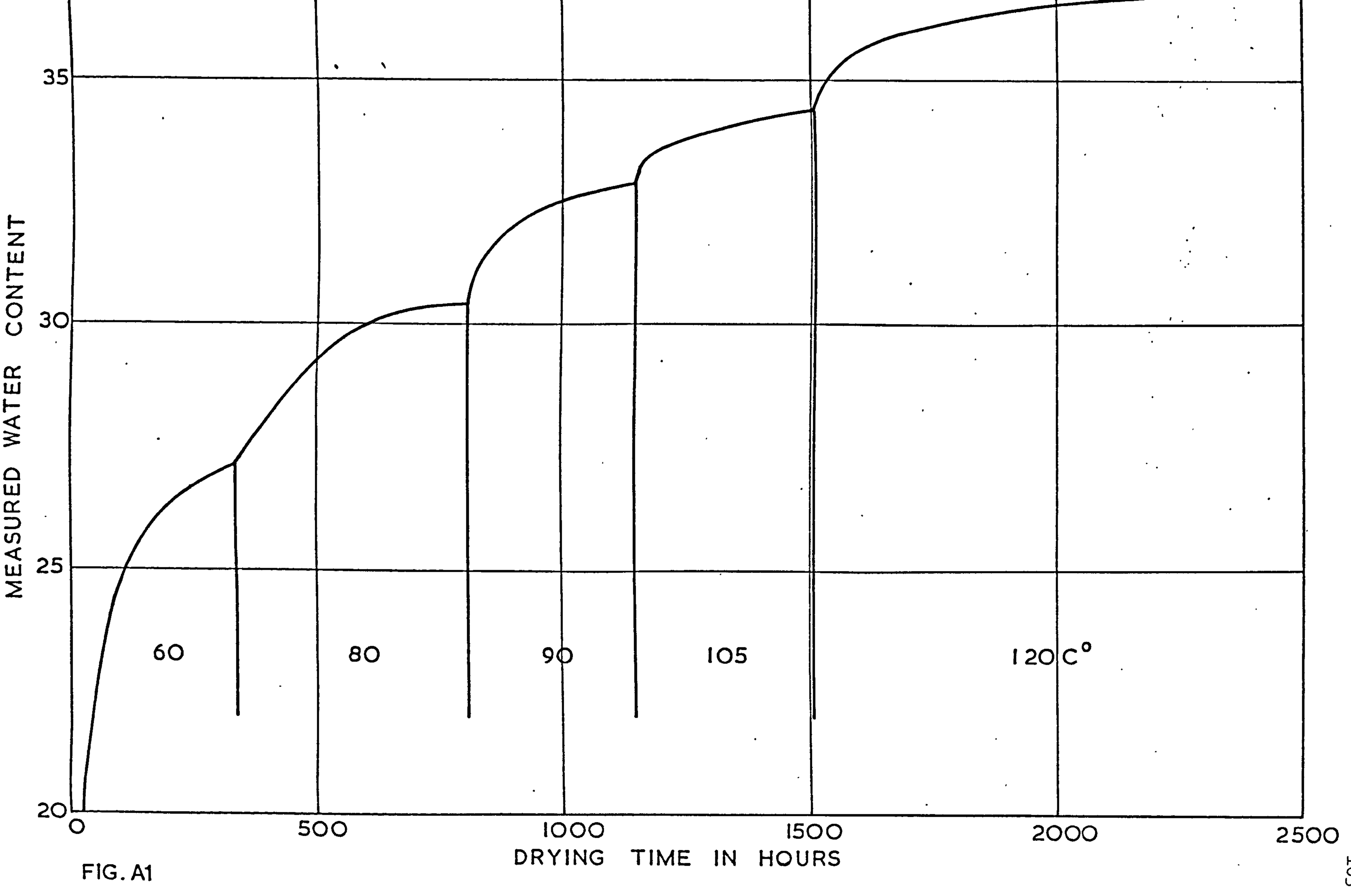

MEASURED WATER CONTENT
35
30
25
20
60
80
90
105
120 C°
0
500
1000
1500
2000
2500
DRYING TIME IN HOURS

FIG. A1

In order to determine the relationship between the weight of water that can be driven off and the initial weight of salt water it was decided to dry some samples of Dead Sea water under normal oven conditions. As drying progressed a skin of salt was formed on the surface of the liquid and this greatly reduced the drying rate. Some brickearth containing a known weight of Dead Sea water was then dried. The drying rate was much faster (the measured water content after 24 hrs was within 1% of that after 1 week) and the values of the ratio:

$$y = \frac{\text{weight of water driven off}}{\text{initial weight of Dead Sea water}}$$

showed very good agreement with the values obtained from the prolonged drying of the Dead Sea water alone at the same temperature. Fig.A2 shows the variation of this ratio (y) with drying time for brickearth and for Dead Sea water.

If the value of y is known for a particular set of drying conditions the initial weight of Dead Sea water present in a sample can be calculated from the measured water content as follows:

If W_s = weight of dry soil

W_l = weight of water driven off

S = weight of salt associated with W_l

i.e. $W_l + S$ = weight of Dead Sea water

M_m = measured water content

M_a = actual water content

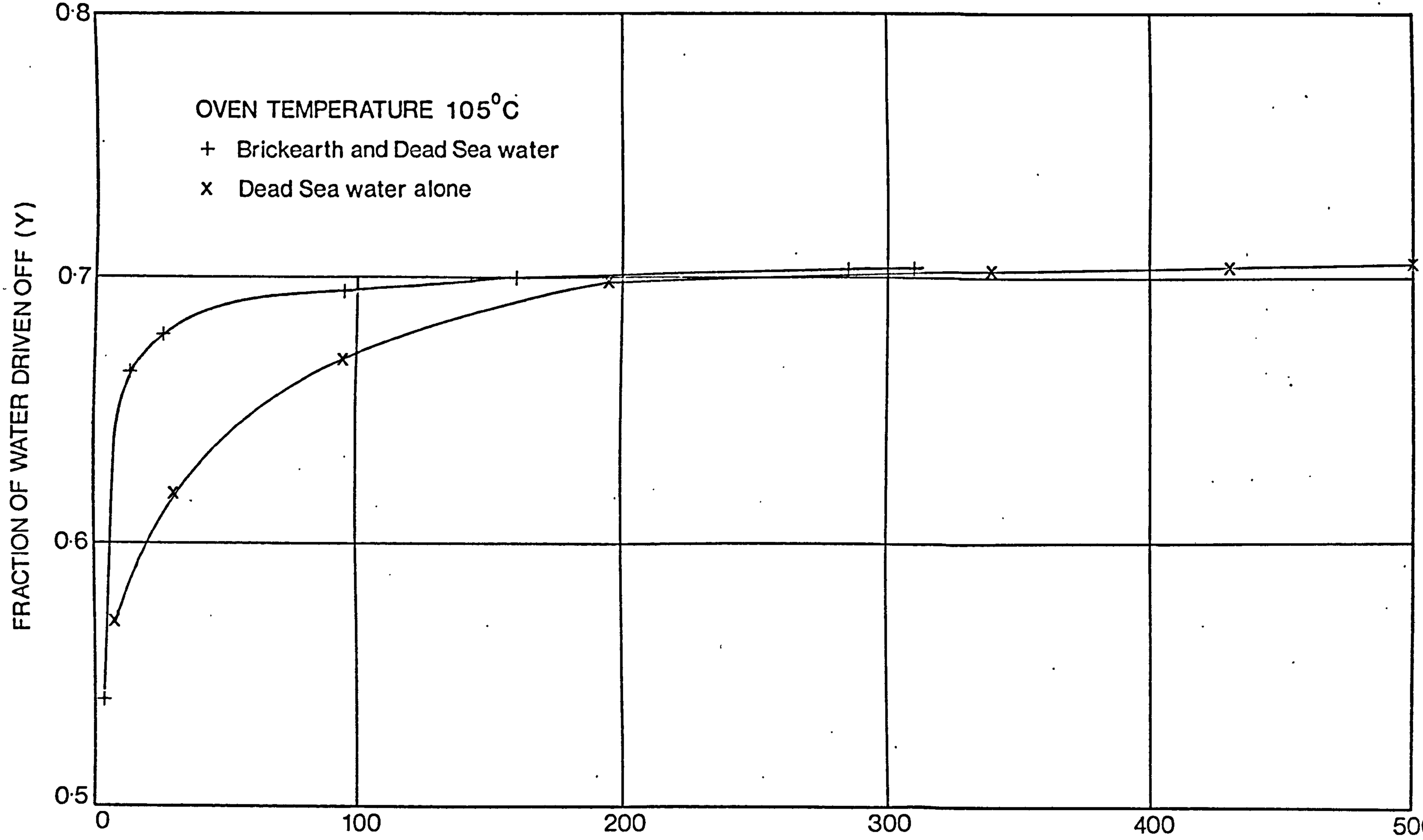

OVEN TEMPERATURE 105°C
+ Brickearth and Dead Sea water
x Dead Sea water alone
FRACTION OF WATER DRIVEN OFF (Y)
0·8
0·7
0·6
0·5
0
100
200
300
400
500
DRYING TIME IN HOURS

FIG. A2

Then the measured water content

$$M_m = \frac{W_l}{W_s + S}$$

whereas the actual water content based on the weight of Dead Sea water present is

$$M_a = \frac{W_l + S}{W_s} \qquad \frac{M_m}{y - M_m(1-y)}$$

where $$y = \frac{W_l}{W_l + S}$$

For example if the measured water content is 24.5% and y = 0.706 (a typical maximum value obtained from drying brickearth and Dead Sea water of 105°C) then the actual water content would be 38.65%.

The value of y to be used for any sample will depend on the drying conditions (time, temperature, oven humidity). For a well ventilated oven in which humidity is not a controlling factor plots of y against drying time can be obtained for different oven temperatures. For small samples which dry rapidly the maximum value of y for the appropriate temperature should be used. For larger samples which dry more slowly the value of y for any drying time must be obtained by drying a similar sample of known water content alongside the samples unless drying to constant weight is employed when the maximum value of y may again be used.

It should be noted that this approach gives the weight of Dead Sea water actually present: relating this to an equivalent fresh-water water content presents a more philosophical problem.

APPENDIX 3 A TABLE OF UNIT CONVERSION FACTORS

Lengths, areas volumes	1 m = 3.281 ft		= 39·37 in
	1 ft = 0.3048 m		
	1 in = 0.0254 m		
	1 m^2 = 10.764 ft^2		
	1 ft^2 = 9.290 x 10^{-2} m^2		
	1 in^2 = 6.452 x 10^{-4} m^2		
	1 m^3 = 35.315 ft^3		
	1 ft^3 = 2.832 x 10^{-2} m^3		
	1 in^3 = 1.639 x 10^{-5} m^3		
Mass	1 kg = 2.205 lb		
	1 lb = 0.4536 kg		
	1 t(tonne) = 1.0 x 10^3 kg		
Shearing resistance τ, or pressure	1 N/m^2 = 0.1020 kg/m^2		= 0.0209 lb/ft^2
	1 kg/m^2 = 9.807 N/m^2		= 0.2048 lb/ft^2
	1 lb/ft^2 = 4.882 kg/m^2		= 47.88 N/m^2
	1 t/m^2 = 1.0 x 10^3 kg/m^2		
Density, γ	1 g/cm^3 = 62.43 lb/ft^3		
	1 lb/ft^3 = 16.02 x 10^{-3} g/cm^3		
	= 16.02 kg/m^3		
	= 1.602 x 10^{-2} t/m^3		
Coefficient of permeability, k	1 cm/s = 1.035 x 10^6 ft/year		
	1 ft/year = 0.966 x 10^{-6} cm/s		
Coefficient of compressibility, m_v	1 cm^2/kg = 7.031 x 10^{-2} in^2/lb		
	1 in^2/lb = 14.22 cm^2/kg		
Coefficient of consolidation, c_v	1 cm^2/s = 3.39 x 10^4 ft^2/yr	= 9.30 in^2/min	
	1 ft^2/year = 0.294 x 10^{-4} cm^2/s		
	1 in^2/min = 0.1075 cm^2/s		

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